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Laboratory Investigation of a Model Multi-tube Closed Conduit Spillway

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LABORATORY INVESTIGATION OF A MODEL

MULTI-TUBE CLOSED CONDUIT

SPILLWAY

BY

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This thesis is approved as a creditable and independent investigation by a candidate for the degree, Master of Science, and is acceptable as meeting the thesis requirements for this degree, but without implying that the conclusions reached by the candidate are necessarily the conclusions of the major department.

Thesis Adviser Date

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INTRODUCTION

Modern hydraulic science is based almost entirely upon experiments. During recent years the empirical approach to the solution of hydraulic problems has gradually been supplemented by a closer adherence to fundamental concepts.

Hydraulic studies, by the use of models of closed conduit flow systems, provide a reliable means of obtaining useful and accurate design data. Closed conduit flow is defined as flow in a system in which the periphery is completely closed.

Interest in the improvement of the operation of closed conduit spillways through the study of experimental models has increased rapidly in the past few years. Studies are no longer directed entirely toward the determination of theoretical aspects of spillway operation, but also are concerned with the immediate problems of design in order to improve performance. The hydraulic laboratory plays a most important role in the design of these structures. The reliability of tests conducted on hydraulic models seems to be universally accepted.

Renewed interest in the hydraulic performance of closed conduits is evidenced by the number of studies reported since 1950. The hydraulics of closed conduit spillways is not as simple as previous thoughts had indicated and is a reason for the renewed interest. Another reason is that many of these structures are being constructed each year and small savings or improvements in each structure generally result in large total savings (8).

The closed conduit spillway is of considerable importance, especially when it is considered that the common culverts used by highway engineers; the "trickle tubes," drop inlet culverts, and uncontrolled pipe outlets used by the Soil Conservation Service for farm and ranch ponds, erosion and flood control dams; and the large shaft or morning glory spillways used on major dams; are all closed conduit spillways (7). The major role played by the closed conduit spillway in the Soil Conservation Service program is apparent.

A mechanical spillway should possess three qualities (12): 1. good hydraulic characteristics, 2. be simple and inexpensive to manufacture, and 3. be easy to install. The designs of mechanical spillways in use today are many, but seldom do they possess all three above qualifications. The drop inlet spillway appears to be best adopted to easy installation and manufacture. The major hydraulic problem is obtaining the maximum rate of flow through a structure with the lowest possible head and still keep the structure economical (12).

Presently, the control of runoff on watersheds in this country is being employed extensively to conserve and to retain runoff. In the past decade, it has become evident that if a desirable, over-all soil and water conservation program is to be attained, the scientific control and management of runoff on small watersheds is a necessity. The development and control of these small watershed areas has yielded invaluable endowments for many rural communities.

With the increase in utilization of detention water, a greater need for prediction of the quantity of surface runoff to be expected from small watersheds is evident. It frequently is found that specific data are not available on runoff from the contributing watershed area. The Agricultural Research Service of the United States Department of Agriculture, established in 1953, is collecting and analyzing runoff data on small watersheds throughout the country. This interest of the Agricultural Research Service in runoff studies provided the incentive for the investigation of this particular closed conduit spillway.

Figure I is a schematic diagram of the installed spillway in the detention dam of Newell Lake, Newell, South Dakota. The spillway consists of a box inlet and three 42-inch corrugated metal conduits which empty into a channel 47 feet below the inlet.

The dam, completed in 1959, is located on South Willow Creek, Butte County, South Dakota, approximately eight miles north of the city of Newell. The pond surface area covers 168.5 acres at the elevation of the inlet of the spillway. The contributing watershed area is 8,666 acres and consists entirely of native range land (14).

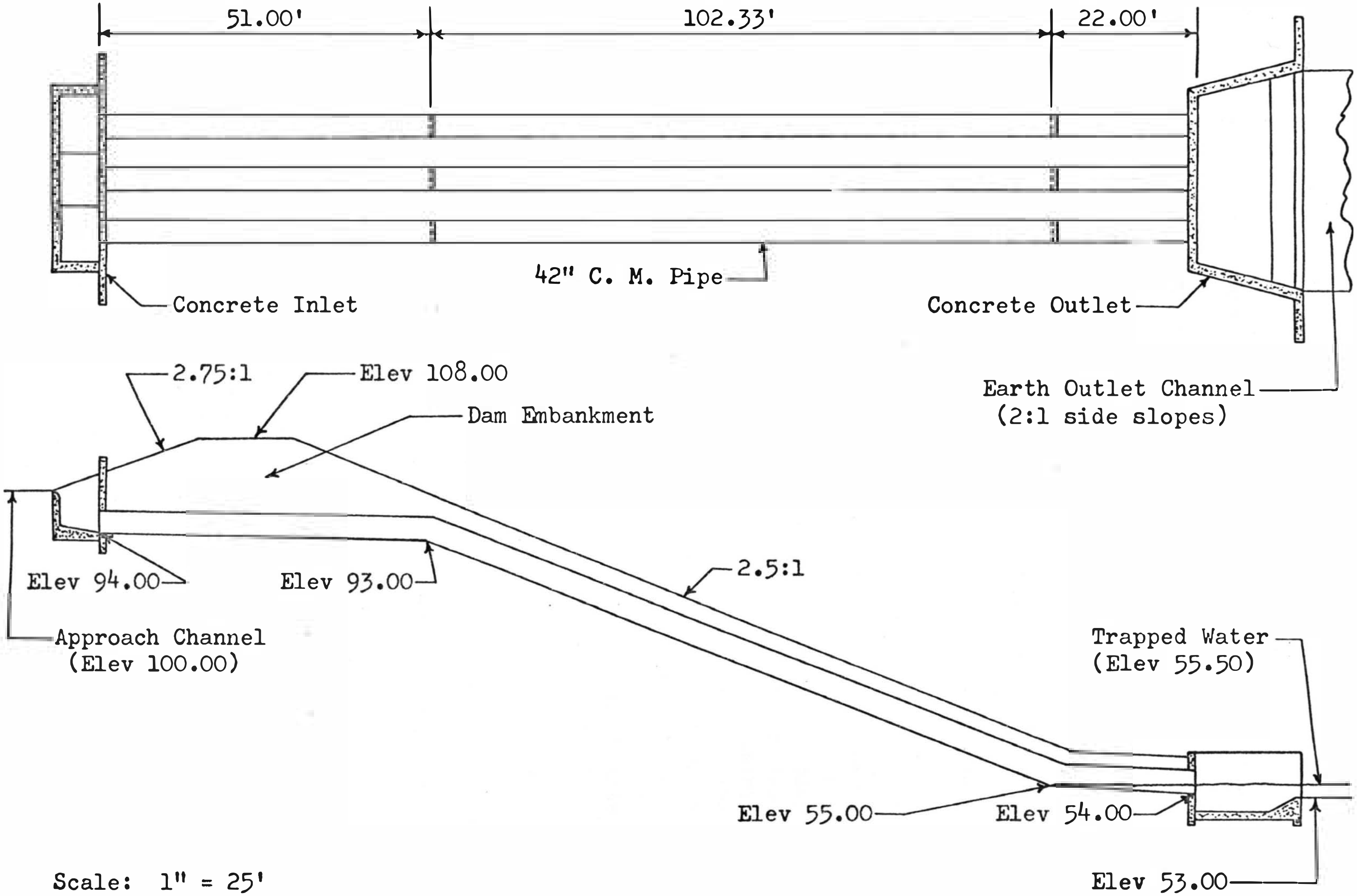


Figure I. Schematic of Spillway

REVIEW OF LITERATURE

During the years 1752 and 1753 the first avowed model experiments were made by an English engineer, John Smeaton (25). The results of these experiments were published in his gold-medal paper of 1759, "An Experimental Inquiry Concerning the Natural Powers of Water and Wind to Turn Mills, and Other Machines, Depending on a Circular Motion." Smeaton's introductory remarks have unconsciously found their way into many a recent paper:

> What I have to communicate on this subject was originally deduced from experiments made on working models, which I took upon as the best means of obtaining the outlines in mechanical inquires. But in this case it is very necessary to distinguish the circumstances in which a model differs from a machine in large; otherwise a model is more apt to lead us from the truth rather than towards it. Hence the common observation, that a thing may do well in a model that will not answer in large. And, indeed, though the utmost circumspection be used in this way, the best structure of machines cannot be fully ascertained, but by making trials with them, when made of their proper size.

The printed version of the lectures of a French engineer, Ferdinand Reech, which were published in 1852, contained a general development of similitude principles based upon Newton's Laws of Motion (25). The following conclusions are taken from there:

> Thus in the event that one had determined by experiment the resistance of a model vessel, or even the complete characteristics of a model steamship with wheels or propellers, in terms of certain known forces, one would only have to build the one or the other model (i.e., the prototype) with linear dimensions l times as great and multiply all the observed velocities by the quantity
>
> $$u = \sqrt{l}$$
>
> for the new systems to function similarly to the earlier one, giving rise to forces of which the static intensities would all be augmented in proportion to the cube of the ratio of the linear dimension . . . .

> With regard to the adherence or friction of a liquid against a smooth boundary, the little that one now knows seems to indicate that the forces of this sort vary, in effect, very nearly as the square of the velocity.

Reech was thus the first to express what is now known as the Froude criterion of similitude, and in France Reech's name is justifiably associated with that of Froude in its designation.

The earliest studies pertaining to closed conduit spillways were those conducted on drop inlet spillways at the University of Wisconsin in 1933, under the direction of L. H. Kessler (8). The gaining of hydraulic knowledge of some of the various forms of drop inlet spillways with desirable hydraulic characteristics was the primary purpose of that study. Poor hydraulic characteristics were found in the simpler square-edged inlet; whereas, the square type morning-glory shaft, more difficult to construct, was found to possess fairly good hydraulic characteristics.

During the 1930's a few field tests were made by plugging the drop inlet, waiting for runoff to fill the reservoir, and removing the plug (8). The barrels of some of these spillways were on a slope steeper than that of the hydraulic friction grade line, yet the barrel flowed completely full--a phenomenon that previously had been thought impossible when the outlet was unsubmerged.

In 1941, Dodge (10) and his associates of the University of Wisconsin checked the validity of drop-inlet model studies by field tests. An experimental field structure and a model on a 1:6 scale were used for the tests. Discharge characteristics were used for comparison of the model and the field structure. The field structure was built of reinforced concrete and the model was made of a combination of wood, plastic and window glass. It was discovered that discharge predictions could be made for the field structure from the model tests.

Experiments on the hydraulics of closed conduit spillways were begun in 1941 at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota, and have been conducted, with interruptions, since that time (8). When barrel slopes were steep and the entrance of the barrel was square-edged, the earlier studies showed that in order to attain full pipe flow the drop inlet must be five barrel diameters high. This drop inlet height was much greater than economically desirable to use for many installations.

In 1950, Blaisdell at the University of New Hampshire conducted studies on square risers and circular barrels (26). Effects of the size of the structure, of the slope, and of the circulation around the headwall, along with the theory for closed conduit spillways, were studied with respect to the head-discharge relationship.

Blaisdell (7) made some tests of a closed conduit model spillway in which the slope of the conduit was as high as 30 per cent. The tests were made at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota in 1953. He found that it was not necessary that the outlet be submerged for the conduit to flow full when the conduit slope was steep. The outlet discharged freely, yet he observed that the conduit may flow completely full. Blaisdell said that some would say this was impossible, their reasoning undoubtedly based on the fact that the greatest capacity of a circular conduit on a steep slope is achieved when the depth of flow in the conduit is 93 per cent of the diameter.

The verification of the model prototype relationship and the perfection of methods of analyzing the test data were the directions of Blaisdell's laboratory tests until about 1949. By 1953, he had this part of the problem well in hand and efforts were then devoted to a study of inlets.

The tests to 1953 (7) had shown that: 1. conduits will flow full when on steep slopes, 2. the Froude Model Law can be used to scale laboratory tests to prototype sizes even though considerable air is sometimes mixed with the water, 3. the flow through the spillway can be computed using the hydraulic laws for weirs and pipes which are already known, and 4. pressures within the spillway can be determined in a model and computed for its prototype even though the conduit length, roughness, or total fall is not reproduced exactly in the model.

Nelson (23) in 1956 was concerned in investigating and classifying the flow regimes of a drop inlet spillway tower. The tests were conducted on a thick-walled tower which was square in cross section and did not have a rounded lip. He found that five flow regimes could be identified at the entrance to the spillway tower: 1. weir flow with clinging nappe, which occurs at low heads, 2. weir flow with aerated nappe, the regime most commonly observed, 3. orifice flow, 4. vortex flow, and 5. full flow.

Some research has been done on a spillway called the hooded inlet, which is considered to have good hydraulic characteristics. The hooded inlet is formed by cutting a pipe at an angle and laying the pipe so the longer side is at the crown or top side. This forms a hood over the pipe entrance. Generally, this spillway is considered to be simple and to have the hydraulic aspect of maximum spillway discharge under a relatively low head.

Blaisdell and Donnelly (6) have done considerable work on the hooded inlet. Their work has been concentrated on finding a hooded inlet which produces greater capacities under relatively low heads of water. Some of the aspects of the inlet studied were: 1. length of hood, 2. conduit slope, 3. vortex inhibitors, 4. wall thickness of the inlet, and 5. approach conditions.

In 1961, Edwards (12) did some investigations on the hooded ogee pipe drop spillway. The goals of his work were similar to those of Blaisdell and Donnelly. His conclusions indicated that the ogee section was an aid in filling the spillway and permitted the use of only one size conduit for the entire spillway, including the riser.

In 1963, Schmer (26) at South Dakota State University conducted studies on the comparison of theoretical, laboratory and field discharge ratings for a closed conduit spillway. In his investigation, Schmer attempted to simulate a corrugated metal section of the spillway by slipping a coil of wire inside Lucite pipe. This method of simulation did not prove to be too satisfactory. Schmer's investigation indicated an amazingly close relationship between the discharge ratings in spite of his corrugated metal simulation.

Many studies have been conducted on closed conduit spillways. Some of these have focused on the inlet and some on the conduit. In general, all have been an attempt to analyze a particular condition and its effect on the over-all performance of the spillway.

OBJECTIVES OF THE RESEARCH

Generally the construction and testing of a model are carried out to substantiate the design of a proposed structure. In this case, a model of an existing field structure was constructed and tested in order to obtain hydraulic criteria of the flow through the structure.

The design of the spillway under investigation is uncommon for a small detention structure. It was the desire of the author to obtain criteria on the spillway design in order to make recommendations as to future use. If this type of spillway is desirable, its use could be beneficial for future small watershed detention basins.

Five objectives were set forth to seek information in order to evaluate the flow characteristics of the particular spillway under study:

1. To obtain a head-discharge curve for the prototype closed conduit spillway.

2. To determine the effect on the spillway performance of the central portion of the conduits, which is on a 40% slope.

3. To determine the effect on the spillway performance of one box inlet for three discharging conduits.

4. To determine if any negative pressures are present, and if so what effect they have on the operation of the spillway.

5. To find a simple and acceptable method of simulating corrugated metal pipe, in conjunction with Lucite pipe, for this particular investigation.

MODEL DESIGN

A model may be defined as a system by whose operation the characteristics of similar systems may be predicted. This definition is general and applies to other than hydraulic models (9). Hydraulic models are generally smaller than their prototypes; in fact, the chief difficulty experienced is in making them sufficiently large. The definition of a model implies nothing as to its appearance, although it is generally assumed to be a small-scale reproduction of the prototype, even though hydraulic models are frequently distorted.

In general, the laws governing the relationship of model to prototype are derived from laws that govern the action of the phenomena under investigation. The operation of, and results obtained from, hydraulic models usually may be transferred to the prototype by the use of model laws which may be developed from principles of similarity. The relationships most commonly used may be expressed in the form of dimensionless groups which by their numerical value characterize the type of flow under consideration. The derivation of the model laws assumes that for equal values of the dimensionless characteristics the corresponding flow patterns in model and prototype are similar (9). The dimensionless groups most commonly used in hydraulic experimentation are designated as Froude's number, Reynolds' number, and Weber's number. They are derived from a consideration of the forces of gravity, viscosity, and surface tension, respectively, in conjunction with the resisting force of inertia.

Principles of Similitude

In a hydraulic-model study, it is desired that the physical behavior of the model simulate in a known manner the physical behavior of the prototype, so that the latter can be predicted from the former. Several kinds of similarity are defined (21): 1. Geometric similarity exists when the ratios of all homologous dimensions on the model and prototype are equal. Thus, geometric similarity involves only similarity in form. 2. Kinematic similarity exists when the ratios of all homologous velocities and accelerations are equal in the model and prototype. Thus, kinematic similarity is similarity of motion. And 3. Dynamic similarity requires that the ratios of all homologous forces be the same in the model and prototype. Thus, dynamic similarity is similarity of the force system.

Dimensional Analysis

During the year 1915, Edgar Buckingham provided the method of dimensional analysis known as the Pi theorem, which has proved a valuable tool for making a more rational appraisal of fundamentals (24). Empirical coefficients are not really satisfactory with today's hydraulic engineer. A deeper insight into the physics of many of the observable hydraulic phenomena is the present goal of scientists interested in this particular field.

A knowledge of the similitude relationship between models and their full-sized counterparts is essential in order to avoid and recognize pitfalls in the use of models, if the necessary accuracy and reliability are to be obtained.

The theory of similitude, upon which model design and analysis is based, may be developed by dimensional analysis. Dimensional analysis is developed from a consideration of the dimensions in which each of the pertinent quantities involved in a phenomenon is expressed (22). The first step in the dimensional analysis of a problem is to decide what variables enter the problem (18). If variables are introduced that do not affect the phenomenon, unnecessary terms may appear in the equation. If variables are omitted that may influence the phenomenon, the calculations may lead to an incomplete or erroneous result. Even though some variables are almost constants (e.g., the acceleration of gravity), they may be essential because they combine with other active variables to form dimensionless products.

In at least 90 per cent of all hydraulic-model studies, the forces connected with surface tension and elastic compression are relatively small and can be ignored safely (29). From a practical standpoint, a particular fluid motion can be represented in a model by considering that either gravity forces or viscous forces predominate.

When gravitational effects predominate in the phenomena under investigation, the pertinent variables are:

D--diameter of pipe
e--roughness of pipe wall
V--average flow velocity
ρ--fluid density
g--gravitational effect
d--average flow depth

Applying the Pi theorem to these variables, it can be shown that the Froude number ($V/\sqrt{gd}$) is the pertinent dimensionless term.

When viscous forces predominate the phenomena under investigation, the pertinent variables are:

D--diameter of pipe
e--roughness of pipe wall
V--average flow velocity
ρ--fluid density
μ--dynamic viscosity of the fluid

Applying the Pi theorem to these variables, it can be shown that the Reynolds number ($\rho VD/\mu$) is the pertinent dimensionless term.

It is essential that both the Froude and Reynolds numbers be the same in the model and the prototype if complete similarity is to be obtained. To satisfy more than one law in any given case, the physical properties of the testing fluid would have to be variable over relatively broad limits. To satisfy both the Froude and Reynolds laws simultaneously, it would be necessary that:

$$\frac{V_m/(g_m d_m)^{1/2}}{V_p/(g_p d_p)^{1/2}} = \frac{(\rho_m V_m D_m)/\mu_m}{(\rho_p V_p D_p)/\mu_p} = 1 \qquad (1)$$

$$\frac{V_m}{V_p} = \frac{(g_m d_m)^{1/2}}{(g_p d_p)^{1/2}} = \frac{\rho_p D_p \mu_m}{\rho_m D_m \mu_p} \qquad (2)$$

$$V_r = (g_r L_r)^{1/2} = \frac{\nu_r}{L_r} \qquad (3)$$

where:

m = model
p = prototype
r = ratio
$\nu = \mu/\rho$ = kinematic viscosity
L = linear dimension

Because the gravitational constant is nearly always the same in model and prototype, that is $g_r = 1$, the kinematic viscosity ratio would have to be related to the length scale as follows:

$$\nu_r = (L_r)^{3/2} \qquad (4)$$

Satisfaction of this criterion for a hydraulic model would require a kinematic viscosity that normally would not be available in a testing fluid.

Because of a limiting factor for physical properties of practical testing fluids, only one similitude law can be satisfied in any given instance. The accepted procedure for applying relations consists of selecting and applying the dominant law.

Usually, when the Reynolds number of the model exceeds a value of 10,000, and depth of flow is substituted for D in the Reynolds number, the viscous forces are relatively unimportant (29).

Velocities encountered in the performance of the spillway under investigation will result in relatively large Reynolds numbers. It can be seen that the slope of a curve is relatively flat when we plot the Reynolds number against the Darcy-Weisbach friction factor (f) for this study. The flatness of the curve indicates that there will be no appreciable change in the Darcy-Weisbach friction coefficient. The Froude number can then be considered the dominant law or parameter and the Reynolds number a secondary parameter.

Model-Prototype Scale Ratio

In this investigation, one of the primary requirements was to simulate the relative roughness of the conduit; therefore, the model-prototype scale ratio must be such that there is similarity between the respective roughnesses of the prototype and model conduits. Both gravity and fluid friction influence the motion patterns of moving particles in this situation.

The Manning formula will give a suitable relation within certain limits of applicability in a case such as this. Thus

$$V_r = \frac{V_m}{V_p} = \frac{\left[\left(\frac{1.49}{n}\right)R^{2/3}S^{1/2}\right]_m}{\left[\left(\frac{1.49}{n}\right)R^{2/3}S^{1/2}\right]_p} \tag{5}$$

Because we have a case of geometric similarity between model and prototype, the values of S_p and S_m are equal. The roughness of the model and prototype are seldom, if ever, equal; therefore,

$$V_r = \frac{(L_r)^{2/3}}{n_r}, \; L_r \text{ being substituted for } R_r \tag{6}$$

This relation has no bearing on the requirements of dynamic similarity, but it does provide the basis for control of the roughness of a model in order to achieve dynamic similarity (2). For such similarity, in accordance to the Froude law for homologous velocities,

$$V_r = (L_r)^{1/2} \tag{7}$$

Equating equations (6) and (7)

$$n_r = \frac{(L_r)^{2/3}}{(L_r)^{1/2}} = (L_r)^{1/6} \tag{8}$$

When gravity is the predominant force, the roughness ratio must equal the sixth root of the linear scale in order to obtain dynamic similarity in the model. In model construction, surface roughness is often controlled in accordance with this relation (2).

Assumptions as to the roughness of the respective materials used to construct the model and the prototype must be made in order to select a proper model-prototype scale ratio. It is generally assumed that the value of Manning's "n" for corrugated metal pipe is somewhere between 0.021 to 0.025 (17). The assumption being if the pipe has a bituminous coating, as is the case under consideration, the roughness value is not affected. A value of n = 0.009 was assumed for Lucite pipe.

With the use of derived equation (8) and the assumed roughness values, the length ratio was calculated to range between 1:160 and 1:450. One can quickly surmise that a length ratio within this range would be impractical for this investigation and would cause the viscous forces to be other than a secondary parameter. The proper procedure to follow, in a case such as this, is to select a reasonable length ratio and to devise a method of pipe roughness for the model in accordance with equation (8).

As outlined by Simmons (27) several general considerations should be taken into account when designing a model. These considerations are: 1. Selection of the testing fluid. Perhaps the most satisfactory, most generally available, and most commonly used is water. Water also is the fluid normally used in the prototype structure. 2. The model should be as large as practicable. Increasing the size of a model usually enhances its usefulness and improves its accuracy. At some point, however, the increase in cost and the difficulty of operation will affect any size advantage. 3. Select a scale ratio to accommodate standard-sized pipes and other model components readily available. Time and unnecessary expense may be saved by such a selection. Odd scale ratios will in no way affect the accuracy of the model or complicate the data analysis. 4. Selection of proper construction material. The materials should combine durability, dimensional stability, ease of construction, and what ever else is deemed important.

After reviewing these considerations, a model-prototype scale ratic of 1:10.5 was selected. This selection would allow the use of 4-inch, inside diameter, Lucite pipe for the conduits of the model spillway.

Conversion Factors

If the flow phenomena are determined primarily by gravitational forces, so that the others (except pressure and inertia) can be neglected, some common conversion factors are:

Velocity: $V_r = (L_r)^{1/2}$

Discharge: $Q_r = A_r V_r = (L_r)^{5/2}$

Pressure: $P = \gamma_r L_r = L_r$, (if same fluid in model and prototype)

Force: $F_r = \gamma_r (L_r)^3 = (L_r)^3$, (if same fluid in model and prototype)

LABORATORY APPARATUS

The hydraulic laboratory used for this investigation was located in the Agricultural Engineering Building at South Dakota State University. The apparatus used in the study was a combination of existing facilities used in similar studies and particular apparatus designed and constructed by the author. Figures II and III are photographs of the complete laboratory test apparatus.

Spillway Construction

The model spillway was basically constructed of a plastic material. The outlet channel below the spillway was constructed of plywood. The spillway itself was designed and cut with precision to within ± 0.05 of an inch for all desired dimensions.

The box inlet of the spillway was constructed of 5/8-inch thick plexiglass. Figure IV is a detailed drawing of the prototype box inlet and Figure V is a photograph of the prototype inlet as installed in the dam. The desired thickness of the weir section of the model spillway was 1.14 inches in order to meet the selected scale ratio. The thickness dimensions of the plexiglass are generally somewhat under the stated thickness and it was found that by utilizing a double thickness an approximation of the required 1.14 inches was obtained. Figure VI is a photograph of the installed box inlet above the tank where the double thickness plexiglass was used. Figure VII shows the inlet below the tank where only one thickness of plexiglass was needed. The spillway was assembled with number 8 by 1-inch screws with silicon waterproof

Figure II. Laboratory Apparatus, Outlet End

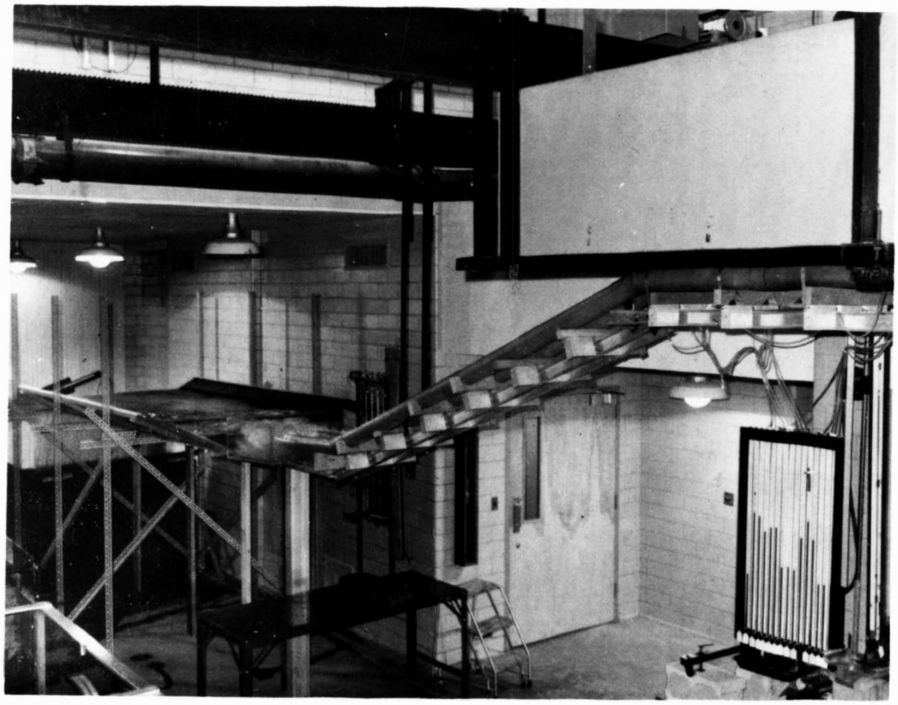


Figure III. Laboratory Apparatus, Inlet End

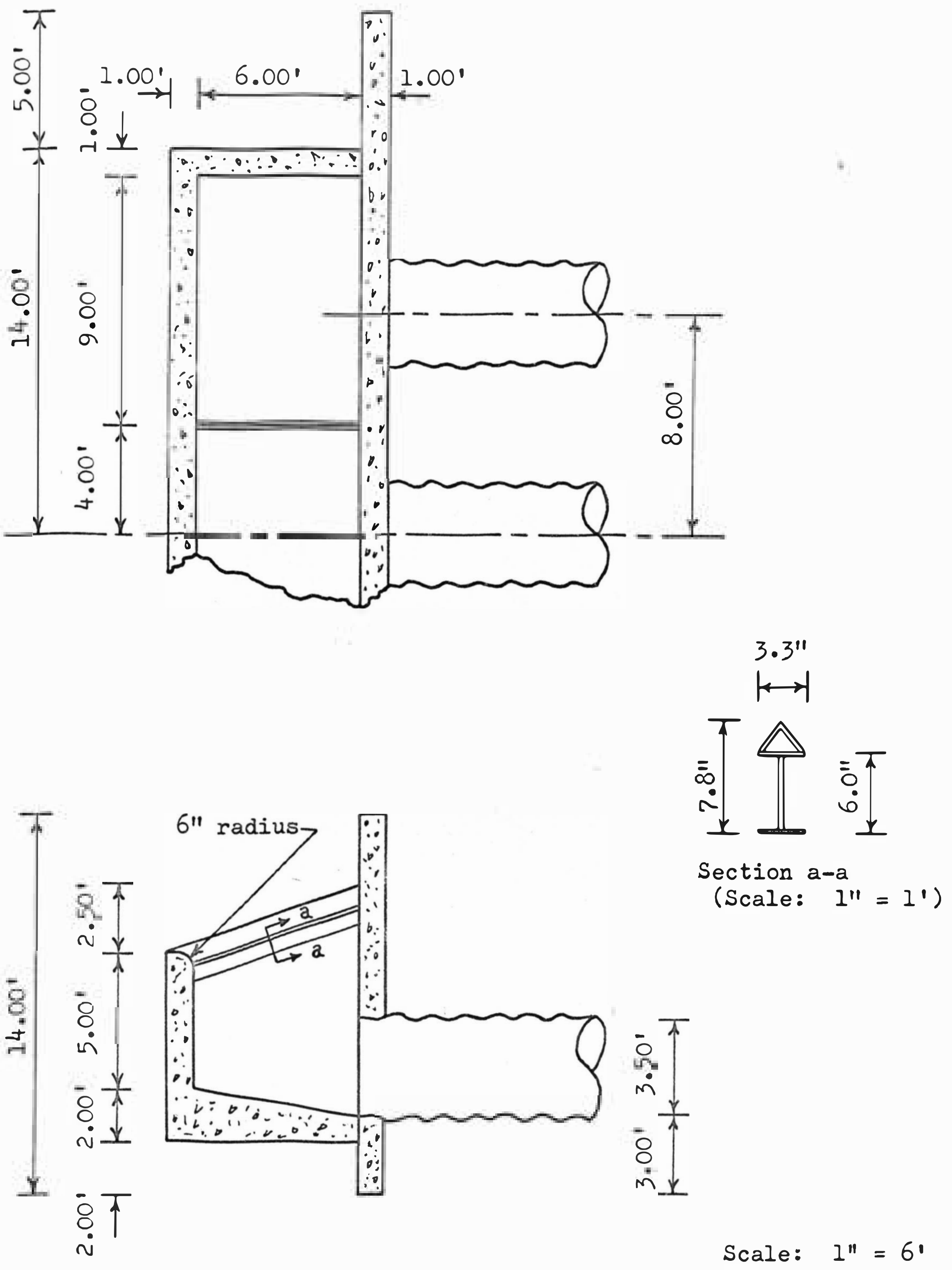


Figure IV. Detail of Box Inlet

Figure V. Prototype Box Inlet

Figure VI. Spillway Section Above the Tank

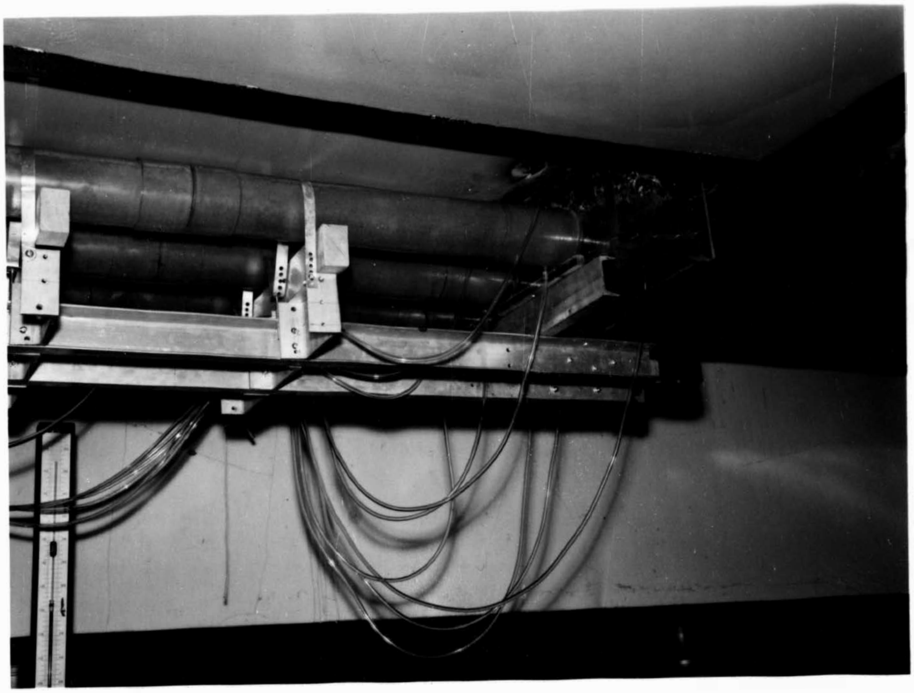


Figure VII. Spillway Section Below the Tank grease in all joining sections. A number 29 drill bit was used to drill the plexiglass, and threads were tapped for each screw. The rounded portion of the weir section was obtained with the use of a jointing mechanism on a table saw plus the employment of some hand sanding. The inlet was supported in the approach-channel tank by a 5/8-inch plexiglass plate which was bolted to the bottom of the tank. The inlet was also supported from the bottom by the spillway conduit cradle. All edges between the spillway, plexiglass plate and tank were sealed with a waterproof caulking compound. All tests were conducted with a 1:2.75 simulated dam embankment which was constructed of 1/4-inch plexiglass.

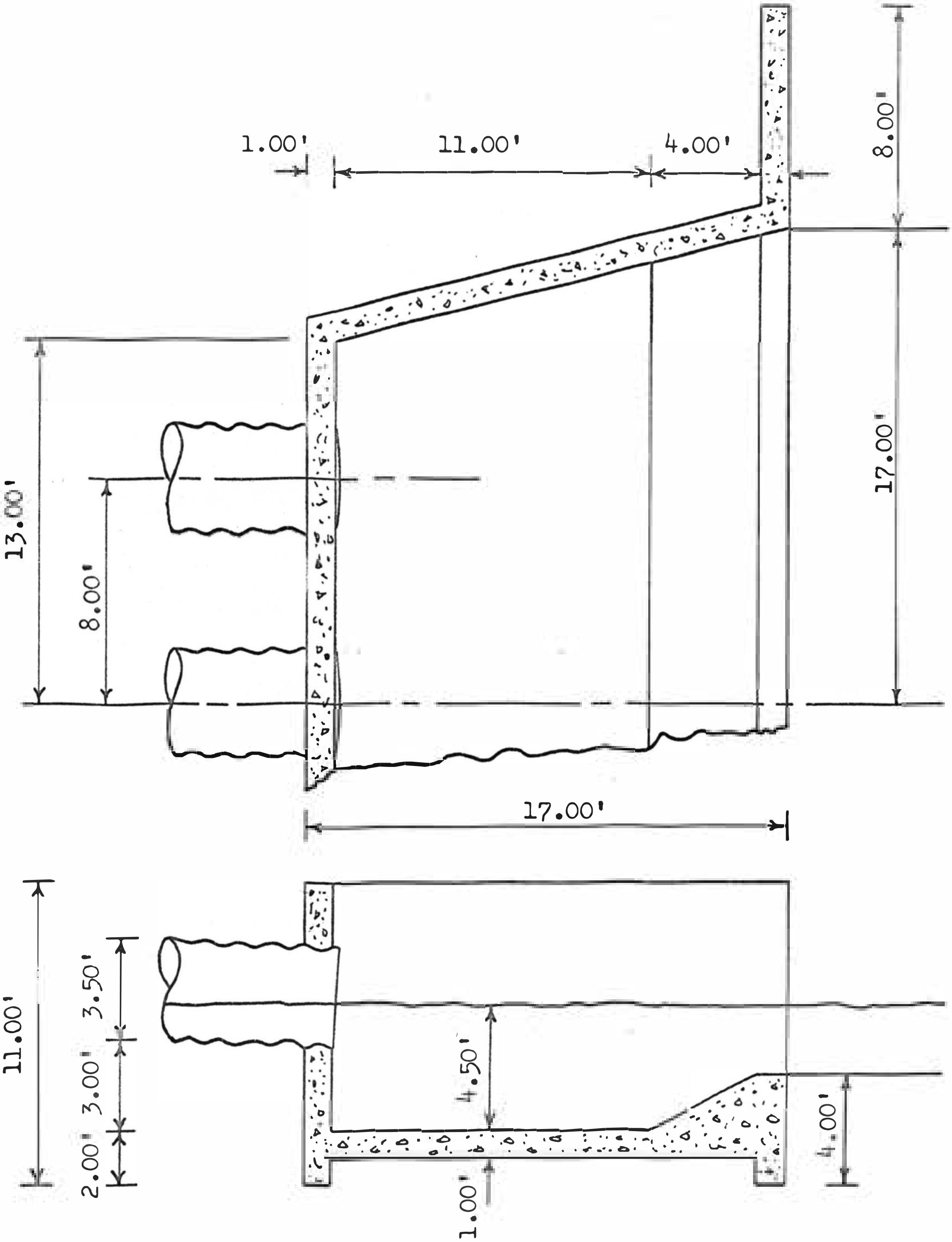
The conduits of the spillway were constructed of Lucite pipe having an average inside diameter of 4 inches and an outside diameter of 4 1/4 inches. The total length of each conduit, from inlet to outlet, was 16.71 feet. The conduits consisted of sections approximately two feet in length with couplings between each section. The couplings were constructed of Lucite pipe 6 inches in length with an inside diameter of 4 1/4 inches. A silicon grease was placed between the conduit and the coupler to protect against any water and air leaks. The first set of couplers was welded to the box inlet of the spillway and the conduits slipped inside to their proper settings. Chloroform was used for all spillway welds. The first two sets of couplers can be viewed in Figure VII. The elbows of the conduits were constructed by cutting the Lucite pipe at the required angle and welding together two such cuts. Figure XVI is a photograph in which one of these elbows can be observed.

The outlet box was constructed similarly to the box inlet. Figure VIII is a detailed drawing of the prototype outlet, and Figure IXa shows the outlet and channel as they exist at the lake site. Figure IXb shows the model outlet and channel. The channel is grass-lined earth with a 34-foot bottom, 1,000 feet long, with 2:1 side slopes. The outlet channel was constructed of 1/2-inch plywood with caulking compound, again sealing all joints and cracks. The outlet end of the channel contained a cross-member of 1/2-inch plywood to simulate the permanent water level in the channel. The channel extended beyond the outlet box for 12 feet.

Conduit Roughness

Considerable time and effort, on the author's part, was spent on this portion of the spillway. Schmer (26) in his study used a coil of wire threaded inside the Lucite pipe and adjusted the coil spacing until the desired friction factor was obtained. This method may have been acceptable for the short section that Schmer had to simulate; but, it was the opinion of the author that this method was undesirable for the particular spillway under study, because there was no practical method of fastening the coils to the wall of the conduit.

The idea of lining the conduit with window screen was adopted. Several sizes and thicknesses of screen and hardware cloth were tested. A final selection of a steel wire, galvanized, 18 by 14 mesh screen was made. Figure X shows a section of the screen used and a section of the



Scale: 1" = 6'

Figure VIII. Detail of Outlet

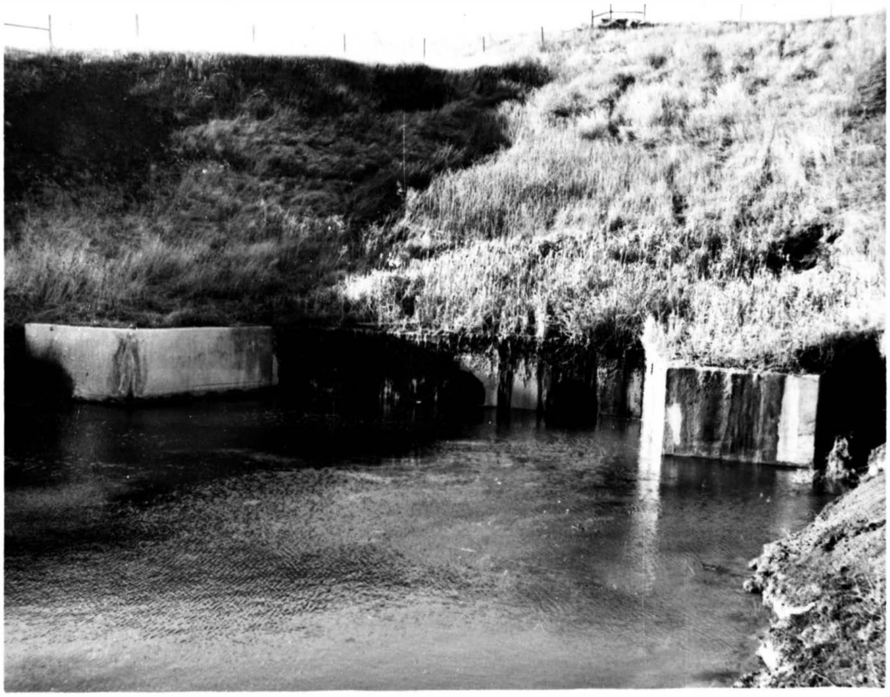


Figure IXa. Prototype Outlet

Figure IXb. Model Outlet and Channel

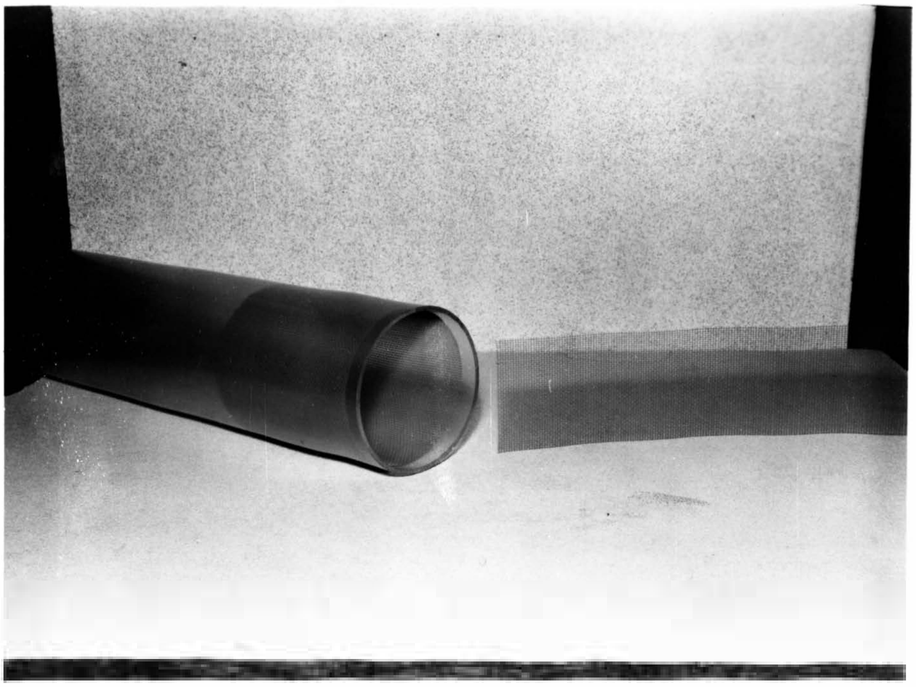


Figure X. Simulated Corrugated Roughness screen as placed in the conduit. The screen was inserted in 10-inch sections and the circumference length was cut to precision through use of foot-squaring sheet metal shears. When a section was cut, it would snap in place tightly against the walls of the conduit and no other method of fastening was necessary.

Approach Channel

The approach channel was a number 10 gage steel tank which had been built for previous studies on closed conduit spillways. (The tank can be seen in Figures II and III.) It was 5 feet wide, 16 feet long, and 30 inches deep. The relatively long tank provided excellent approach channel conditions. The tank was supported from four 12-inch, 16-pounds-per-foot I-beams by four 4-inch, 22-pounds-per-foot channel hangers (12). The tank was so designed that even settling would take place when it was filled with water.

Water was supplied by two 3,900-gallons-per-minute turbine pumps to a constant head tank in the laboratory. From the head tank, water was distributed throughout the laboratory by an 8-inch supply line and then on to the approach-channel tank by a 6-inch supply line. Water entered the tank from its upstream end and from the top. The water was stilled by placing a 6-inch wide, crushed rock barrier across the tank in front of the supply outlet. The spillway flow rate was regulated through the use of valves throughout the supply lines.

Spillway Cradle

The spillway conduits were supported on an existing cradle which was modified to fit the needs of the study. The cradle consisted of two 3-inch aluminum channels, with 6-inch aluminum channel spacers approximately every 18 inches between the 3-inch channels, to support the middle conduit. Six-inch aluminum channel spacers were located on the outside portion of the 3-inch channels, exactly opposite the inner spacers, to support the outer conduits. (Figure VII, showing the conduits and box inlet below the tank, will help clarify the above explanation.) The existing cradle was cut into three sections. The three sections were joined at the required angles to fit the conduit slopes of the model spillway. (Figure III shows the three slopes of the spillway conduit.) The top end of the conduit cradle was bolted to an adjustable channel hanger that was fastened to the center tank hanger just behind the box inlet. The cradle was supported by an adjustable angle-iron hanger bolted to the end of the approach-channel tank just in front of the upper set of elbows of the conduits. The lower end of the cradle was supported by an adjustable, overhead, screw jack that raised and lowered the cradle. The outlet end of the conduits and the outlet box were supported by a 6-inch aluminum I-beam that rested on the concrete floor of the laboratory. The outlet channel was supported by a frame built with stamped, Amco steel angles plus three 2 by 6-inch wood supports from the floor of the laboratory to the centerline of the channel. (All features of the cradle and its supports can be observed in Figures II, III, and VII.) The spillway was supported on the cradle by wood-block spacers between the conduits and the cradle and aluminum straps wrapped around the conduits and bolted to the cradle. All elevations were adjusted by use of a surveyor's level and were set in reference to the spillway lip.

Discharge Measurements

Discharge measurements were made through the utilization of measuring devices located in the laboratory. Low flows were measured with a 90° V-notch weir which was located in a collection box and short channel at the end of the spillway outlet channel. Figure XI shows the low flow measurement device. The outlet channel discharged into the collection box from which the water was permitted to flow around baffles and into the short channel which contained the weir and hook gage.

High flows were measured by 4-inch and 6-inch orifice meters. The orifice meters were connected to a manometer containing a fluid with specific gravity of 1.75. A few of the very high flows were measured with the 6-inch orifice meter connected to a mercury manometer. Both orifice meters were calibrated in the hydraulics laboratory before the closed conduit spillway tests were made.

Stage Recordings

Water stages in the approach-channel tank were recorded by a Stevens Type F Recorder. The recorder had a 1:1 gage scale ratio and the chart drum was activated by a 12-inch float. By using a 12-inch float it was found that the recorder was accurate to ±1 one-thousandths of a foot. A hook gage was employed as a check against the recorder.

Figure XI. Discharge Measuring Device for Low Flows

Figure XII is a photograph of the stage recording instruments. The pen on the recorder was driven by a synchronous motor clock with a time scale of 57.6 inches per day. Figure XIII is a sample of the stage recordings made during the testing of the spillway.

Pressure Measurements

Piezometer taps were located at various points along the bottom of the conduits to determine pressures and to evaluate the piezometric grade line. Piezometric taps were also located on the inlet section of the spillway to obtain any additional knowledge of the over-all spillway performance. Beyond the entrances of the conduits, taps were located only on the center and right conduits. Figure XIV shows the location of all piezometer taps of the spillway. The taps were constructed by securing, with chloroform, a 1/4-inch plexiglass tube, 1-inch long, into the walls of the conduits and box inlet. The tubes had an inside diameter of 1/8 inch. A soft transparent plastic tubing with a 1/4-inch inside diameter was used to connect the taps to the manometers.

The pressures for taps labeled 1 through 5 and tap T were measured by an open-air well, 15-tube, manometer board. The open-air well was attached to a reference manometer by means of a T-system. Figure XV is a photograph of the manometer board and reference manometers with labeled tap locations. The T-system, between the board and reference manometer, contained a valve so that the part of the system containing the manometer fluid could be evacuated of any air that was

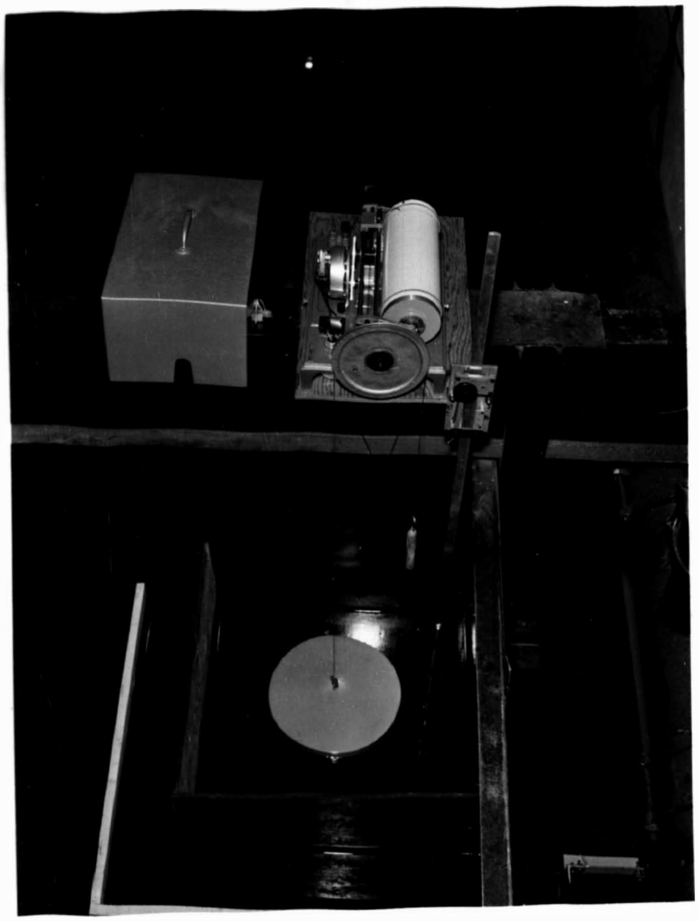


Figure XII. Stage Recorders for Approach Channel

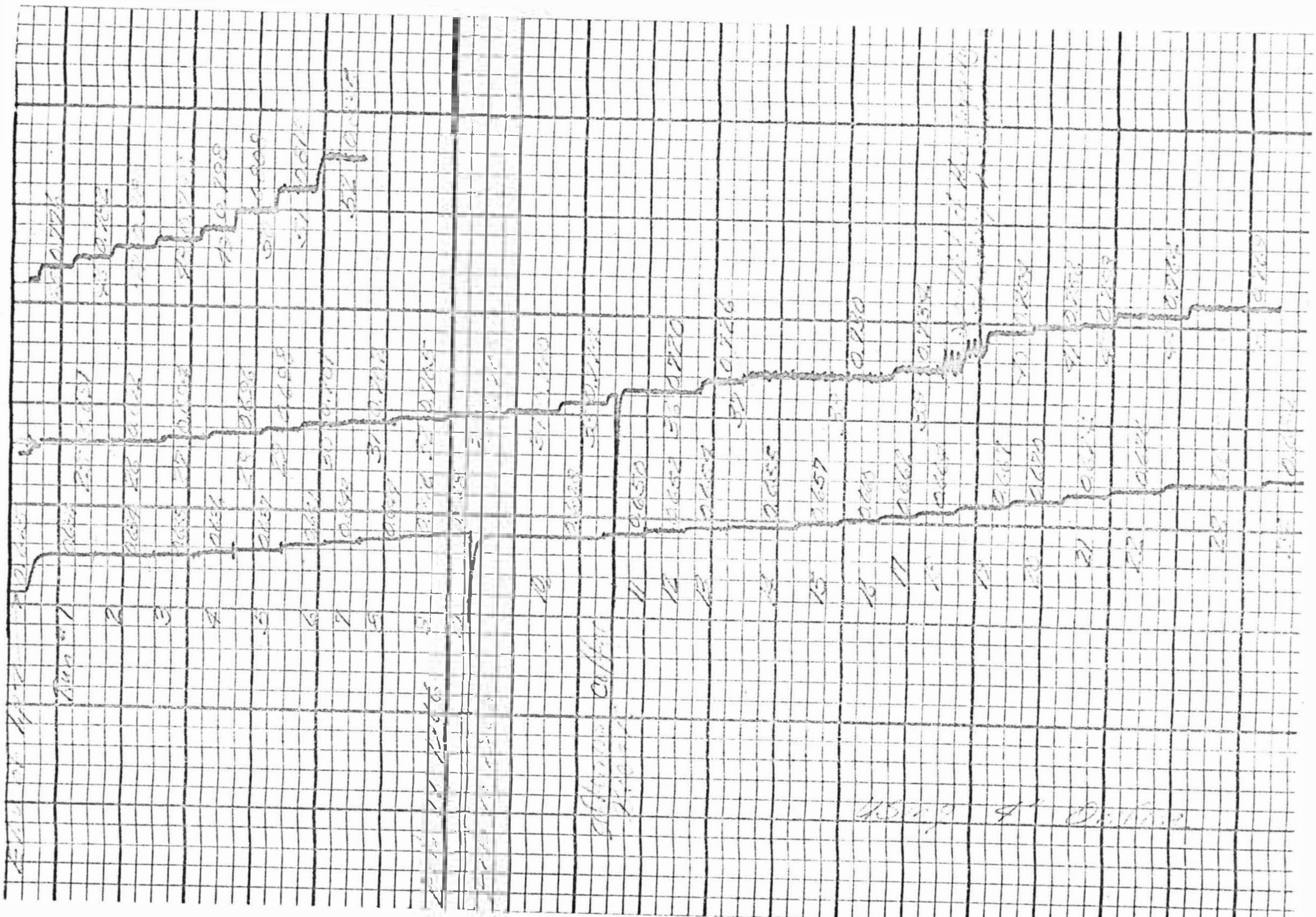
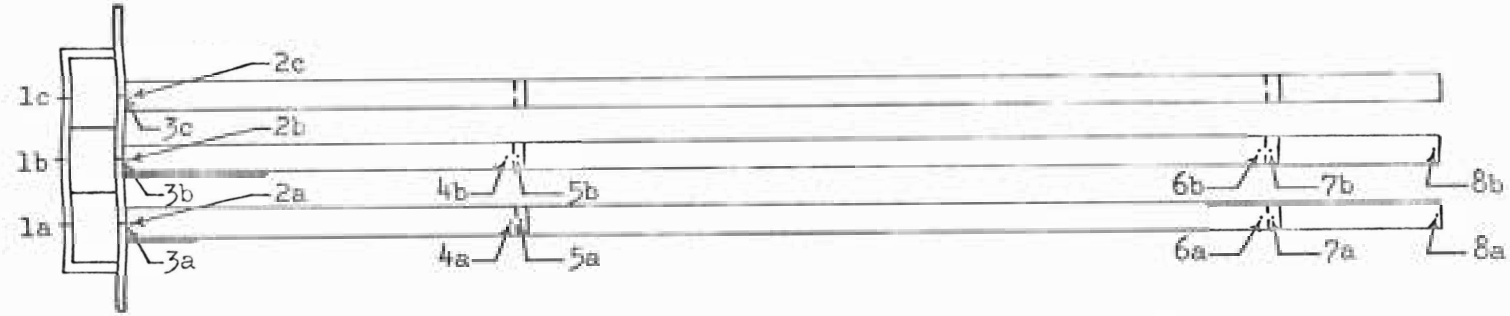


Figure XIII. Sample of Model Stage Recording



Tap No.	Distance from Exit
1a,b,c	Spillway Lip
2a,b,c	50.24D
3a,b,c	Spillway Front Wall
4a,b	36.57D
5a,b	35.41D
6a,b	7.20D
7a,b	5.53D
8a,b	0.46D

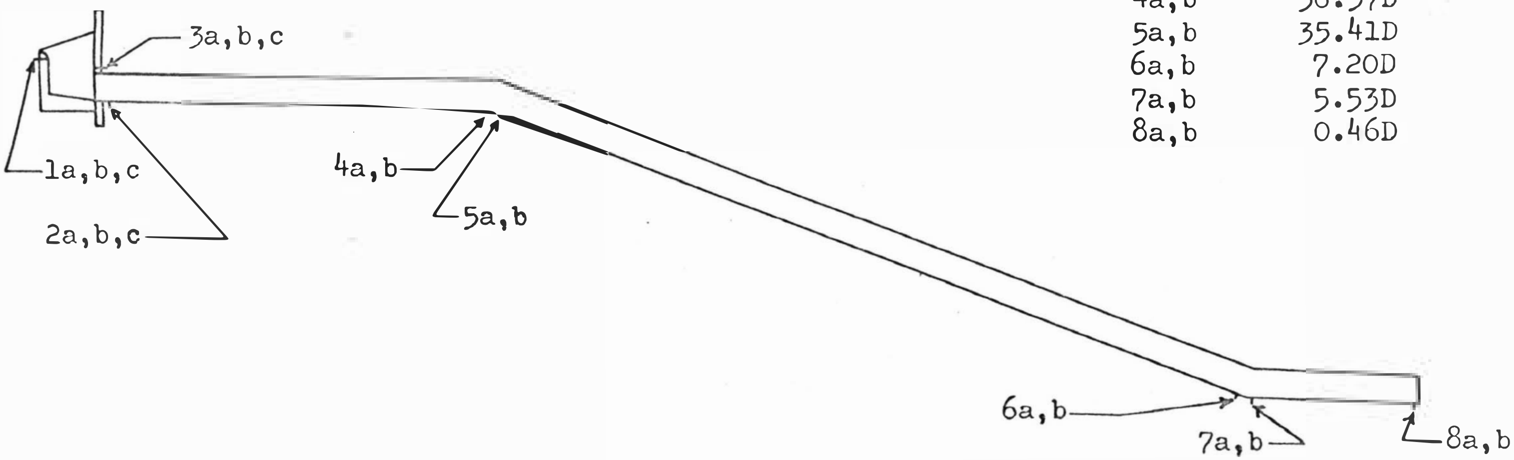


Figure XIV. Piezometer Tap Locations

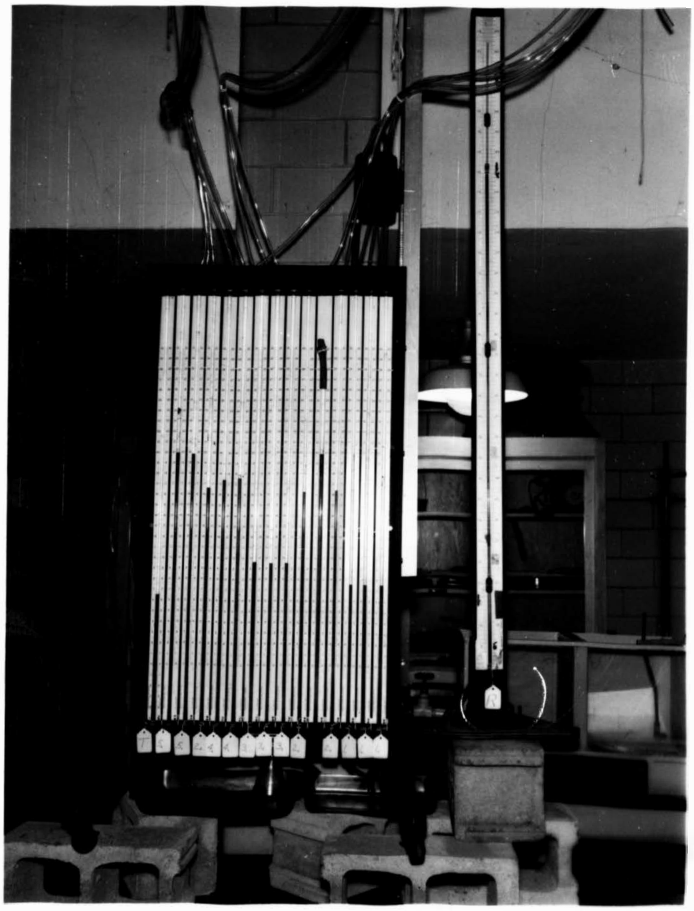


Figure XV. Manometer Board

trapped during filling. A manometer fluid with a specific gravity of 1.75 was used for all tests in the manometer board set-up.

The datum plane of the manometer board was established by using a surveyor's level to run a set of levels from the spillway lip to a specific elevation on the manometer board.

The pressure taps labeled 6 through 8 were connected to differential water manometers. Figure XVI shows the set-up of these manometers. The water in the manometers was colored with red food coloring so the water levels could be read more easily.

LABORATORY INVESTIGATION

Friction Study

Before the performance of the spillway was observed, a study was made to determine a method of obtaining a frictional resistance for the conduits. Two of the conduits on the spillway were disconnected and the tests for friction were conducted on the third conduit. Two piezometric taps were located in the conduit at an interval of three feet. After approximate friction tests were made, the conduit was lined with the selected screen. Final tests were then made by allowing the headpool elevation in the approach-channel tank to stabilize at a level where there was full pipe flow. The discharge was then recorded and the pressures at the two taps were recorded.

The selected screen allowed for a complete lining of the conduits and gave a roughness coefficient which corresponded to a Manning's "n" of approximately 0.022 for the prototype. Table 1 is a summarization of the final test results on the selected roughness simulation. The maximum thickness, as measured with a micrometer, of the screen was found to be 0.025 inches. After inserting the screen into the 4-inch Lucite pipes, the diameter of the model conduits was reduced to 3.95 inches. This reduced the prototype conduit size to a diameter of 41.5 inches under the 1:10.5 scale ratio that was used. The actual diameter of a nominal 42-inch diameter corrugated metal pipe is approximately 41.5 inches.

Table 1. Results of Final Corrugated Metal Simulation Tests

Run (No.)	Q (cfs)	V (fps)	Pressure Loss, (ft)	Elevation Loss, (ft)	Total Head Loss, (ft)	f
100	0.290	3.41	0.038	0.059	0.097	0.059
101	0.295	3.47	0.038	0.059	0.097	0.057
102	0.310	3.65	0.038	0.059	0.097	0.052
103	0.335	3.94	0.054	0.059	0.113	0.052
104	0.325	3.82	0.042	0.059	0.101	0.049
105	0.290	3.41	0.038	0.059	0.097	0.059
106	0.300	3.53	0.038	0.059	0.097	0.055
107	0.285	3.35	0.029	0.059	0.088	0.056
108	0.285	3.35	0.033	0.059	0.092	0.058
109	0.290	3.41	0.050	0.059	0.109	0.066
110	0.300	3.53	0.046	0.059	0.105	0.060
111	0.310	3.65	0.046	0.059	0.105	0.056
112	0.320	3.77	0.050	0.059	0.109	0.054
113	0.290	3.41	0.046	0.059	0.105	0.064

Spillway Study

The laboratory testing and data collection of the spillway were accomplished during eight different periods of several scheduled days. The periods ranged in time from two to six hours. The manner in which the investigation was conducted allowed the author to complete the study without additional personnel.

The procedure used in collecting the data was to set a rate of flow, wait until the headpool elevation stabilized, and make the needed observations and recordings. For weir flow, the headpool stabilized rapidly and successive runs could be made in relatively quick succession. For full pipe flow a relatively long time was required for the headpool to stabilize; but because the number of test runs required in this area was few, the procedure was carried out in the described manner.

Initiation of the testing was accomplished by filling the headpool tank, closing off the water supply, and allowing the headpool elevation to stabilize at the spillway lip elevation. Each successive test was begun by increasing the discharge by a small increment over the previous test. After the headpool elevation had stabilized, the discharge reading was recorded, then the manometer board, reference manometer, and water manometers were read and recorded, the discharge reading was checked, and the headpool elevation was checked with the hook gage for comparison with the stage recorder.

The very low spillway discharges were measured with the 90° V-notch weir and hook gage located at the end of the spillway channel.

The low discharges were measured with the weir and the 4-inch orifice in the supply line. By utilizing both the orifice and the weir for this area of flow, a comparison of the readings could be made for the discharge. The high discharges were measured with the 4-inch orifice and the very high discharges were measured with the 6-inch orifice in the supply line.

Weir flow prevailed until the headpool level was approximately 0.62 of a conduit diameter of depth above the elevation of the spillway lip. Just above this depth of flow the center conduit attempted to flow full. Full pipe flow of all three conduits occurred just below one conduit diameter of depth, approximately 0.99D, above the spillway lip. Between the depths of 0.62D and 0.99D, slug flow prevailed in one or more of the conduits most of the time. There was a considerable section, in this depth area, in which air was carried through the spillway during the time traveling hydraulic jumps or slugs filled the conduits and traveled through them. A more detailed discussion of the spillway flow will be made in the "Results of Tests" section.

ANALYTICAL METHODS

Two hundred twenty-seven test runs were made on the spillway. The headpool elevation was raised to a maximum height of 1.47D above the spillway lip. This elevation carried the discharge of the spillway well into the full pipe flow range. The maximum headpool elevation was also slightly above the elevation of the prototype emergency spillway channel.

Stage Recordings

The Stevens Type F Recorder made a continuous recording of the headpool elevation. (Figure XIII is a sample of the stage recordings showing test runs 1 through 52.) The time scale was such that the smallest horizontal increment was equal to 2.5 minutes. This was not really important because its only use was to obtain a travel speed of the pen that would give a clear headpool elevation recording. The stage scale was such that the smallest vertical increment on the chart was equal to 0.01 of a foot. A check was made of the chart recordings against recorded elevations made with a hook gage. By comparing the two stage recordings, it was possible to analyze the headpool elevations to the nearest ± 0.001 of a foot until very high flows were encountered and analysis was made to ± 0.002 of a foot.

Discharge Readings

Discharge measurements made with the V-notch weir were computed by use of a table, for right-angled V-notch weirs, listed in a handbook of hydraulics by King (16). The formula that had been used for compiling the table was

$$Q = 2.52H_v^{2.47}$$

where

Q = discharge in cfs
H_v = head above V-notch

The discharge readings taken with the weir were recorded to the nearest 0.001 of a cubic foot.

Discharge measurements made with the 4-inch and 6-inch orifice meters were recorded to the nearest 0.005 of a cubic foot. The discharge measurements in which both the V-notch weir and the 4-inch orifice were used were in complete agreement.

Manometer Readings

The manometers were read immediately after the headpool elevation stabilized for a particular test run. Manometer readings were recorded only during test runs where it was thought possible to obtain information on the spillway. (Figure XIV shows the location of the piezometric taps.) Table 2 is a summary of the relative elevations of these taps. Whenever any of the spillway conduits attained slug flow or full pipe flow, the pressures from the entrance to somewhere beyond the first elbow of the full conduit were negative pressures.

Pressures of the manometer board were converted to water pressure by multiplying the fluid differential (S.G. = 1.75) by 0.75. Pressures of the water manometers could be read directly in inches of water. Tubes labeled 1a, b, and c measured pressures of the spillway lip and were sometimes negative. Tubes 2a, b, and c measured pressures

Table 2. Relative Elevations of Model Spillway

Location	Elevation
0.000 Man. Board	0.000
Box Inlet Lip	6.200
Piz. taps 1a,b,c	6.138
Inlet Pipe Invert	5.629
Piz. taps 2a,b,c	5.626
Piz. taps 3a,b,c	6.025
Piz. taps 4a,b	5.537
First Set of Elbows	5.533
Piz. taps 5a,b	5.460
Piz. taps 6a,b	2.015
Second Set of Elbows	1.915
Piz. taps 7a,b	1.903
Piz. taps 8a,b	1.827
Outlet Pipe Invert	1.820
Bottom Outlet Channel	1.725
Channel Water Surface	1.965
Piz. tap T	Approach Tank

just beyond the entrances of the conduits. Tubes 3a, b, and c measured the pressure on the box inlet wall just above the conduit entrances. Tubes 4a and b measured the pressure just before the first elbow and 5a and b measured pressures immediately after the bend of the elbow. Tube T of the manometer board measured the headpool level. Water manometers 6a and b, 7a and b, and 8a and b measured the pressure just before the second elbow, immediately after the second elbow, and just before the conduit exit, respectively.

Grade Lines

The piezometric grade line elevations were computed from the data obtained with the manometers. Typical grade line drawings are shown in Figure XVII.

The piezometric, total energy, and friction grade line and the entrance and elbow loss coefficients were computed from the manometric data.

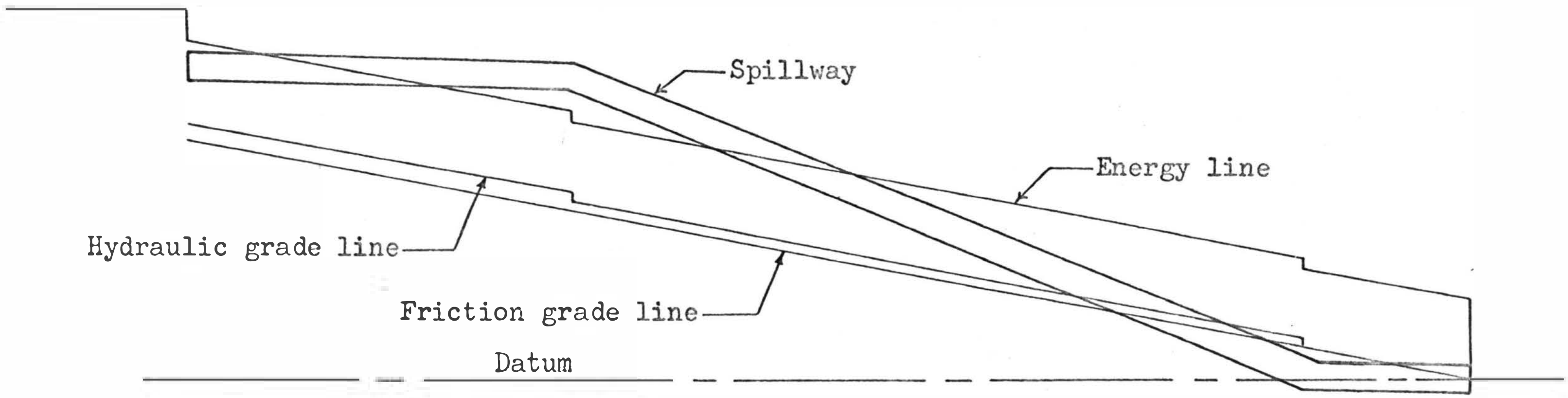


Figure XVII. Typical Grade Lines for Full-pipe Flow

RESULTS OF TESTS

The primary objective of this investigation was to obtain a field head-discharge curve for the specific spillway. Several secondary objectives were included to determine if the spillway is feasible and practical, and to reveal possible problems.

Data, analysis and discussions are found in the following respective sections.

General Spillway Performance

The desirable weir and pipe controls governed the head-discharge relationship for the spillway under study. The weir portion of the rating curve contained a considerable section of slug flow and flow of an air-water mixture.

The conduits of the spillway were at part full flow until a head of 0.62D above the lip of the spillway was attained. At this point the three conduits were full of water up to the first elbow, and the center conduit attempted to run completely full to the outlet. The center conduit would flow full past the first elbow, would draw a slug of air, and a hydraulic jump would travel through and out of the conduit. As the head was increased slightly, the frequency of the jumps increased and a continuous jump was formed just beyond the lower elbow of the conduit. Increasing the flow still more caused several jumps to be in the barrel at one time and they again would move out the end of the conduit. As the flow was continually increased the conduit eventually was filled with a mixture of water and air. Finally, at a head of

0.73D, the water flow became so great that the air flow stopped, and the conduit was completely filled with water. During this sequence of events in the center conduit, the two outer conduits were running full to the first elbow and partly full from the first elbow to the exit.

Full conduit flow of the center tube and part full flow of the outer tubes continued until a head of 0.77D was reached. At this stage of flow one of the outer conduits would draw occasional slugs of air and form a traveling hydraulic jump. There was no way to predict which of the outer pipes would begin doing this. During some of the runs the left conduit would begin this drawing of air and during others the right conduit would do so. During one of the runs the two outer conduits drew slugs of air simultaneously, then the left conduit changed to part full flow, and the right conduit continued to draw air. While a slug of air traveled through one of the outer conduits, the water flow of the center tube would change to an air-water mixture. When the flow reached the point where several jumps were traveling through an outer conduit, the flow in the center conduit would change to flow with traveling hydraulic jumps and the flow of these pipes would be very similar. The two conduits then followed the same flow pattern as did the center conduit through its first slug flow process, and reached full pipe flow at a headpool elevation of 0.87D.

The center conduit and one of the outer conduits continued with full pipe flow, and the third conduit was full to the first elbow and partly full beyond to the exit, until a headpool stage of 0.90D was reached. At this point the third conduit began its cycle of slug flow, changing from an occasional traveling hydraulic jump, to several jumps in the conduit at one time, to an air-water mixture flow, and to full pipe flow. The only change in flow of the center conduit was a slight air-water mixture flow when the third conduit started slug flow. The first outer conduit did have some hydraulic jumps moving through it during the early stages of slug flow in the second outer conduit. The center and first outer conduit returned to full pipe flow when the third conduit contained flow in which several jumps were moving through the pipe. Full pipe flow of all three conduits was attained at a head-pool elevation of 0.99D above the spillway lip.

Figures XVIII and XIX show the prototype spillway in operation and Figures XX and XXI show the model spillway in operation.

The water in the exit channel seemed to have no effect on the over-all discharge performance of the spillway. The purpose of the trapped water is for energy dissipation. Inspection of Figure XXII shows that silt is being deposited some distance beyond the outlet. The silt deposit could eventually cause the conduit outlets to be submerged, which might have an effect on the performance of the spillway.

Spillway Capacity

The spillway rating curve is of extreme significance since the prototype spillway is to be used for measurement of run-off studies of the contributing watershed area. Figure XXIII is a head-discharge curve of the data obtained from the laboratory model. Figure XXIV is a head-discharge for the prototype spillway.

As was previously stated, only the desirable weir and pipe

Figure XVIII. Flow into Prototype Spillway

Figure XIX. Flow from Prototype Outlet

Figure XX. Flow into Model Spillway

Figure XXI. Flow from Model Outlet

Figure XXII. Prototype Outlet Channel

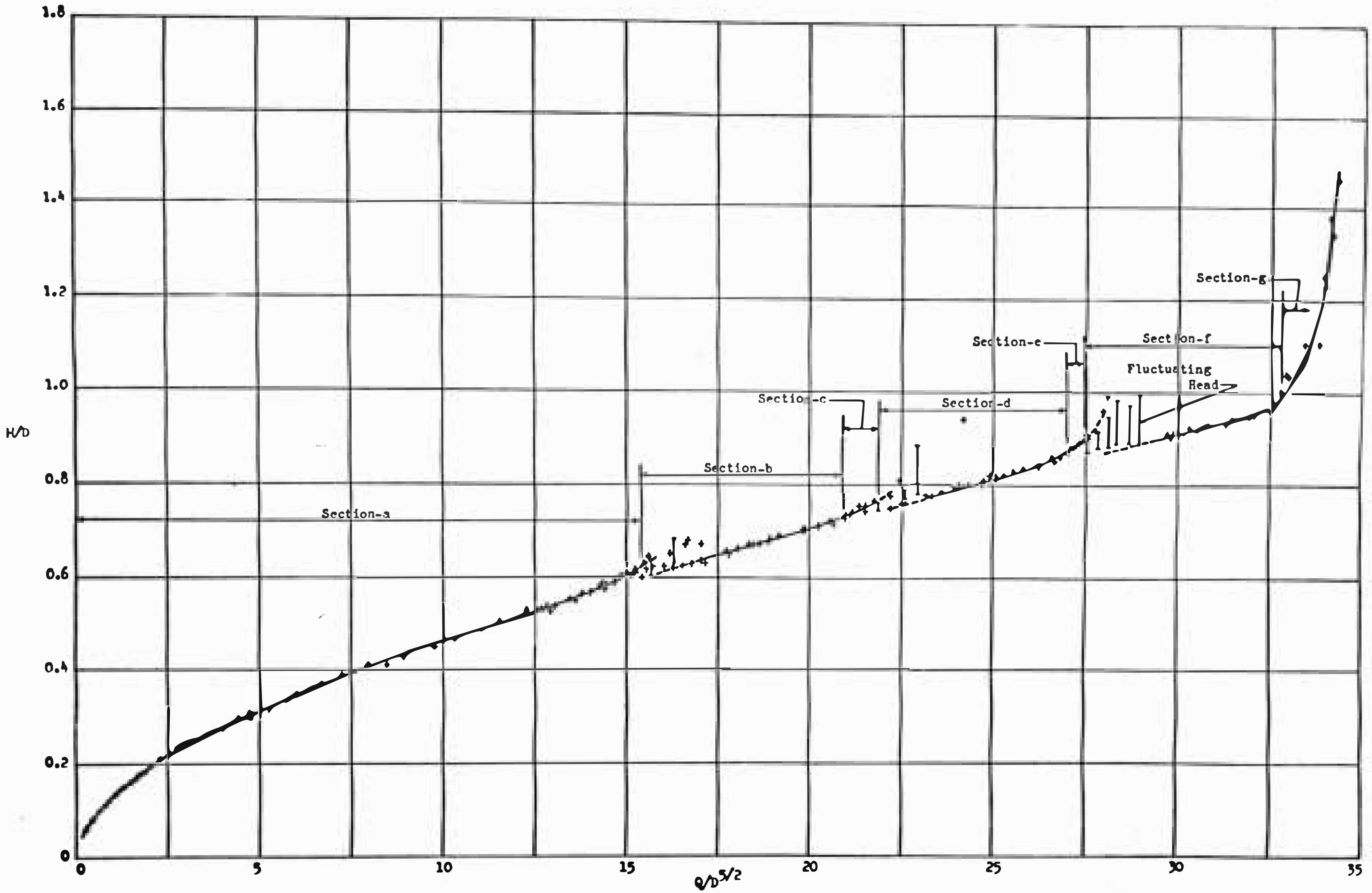


Figure XXIII. Head-Discharge Curve From Model Data

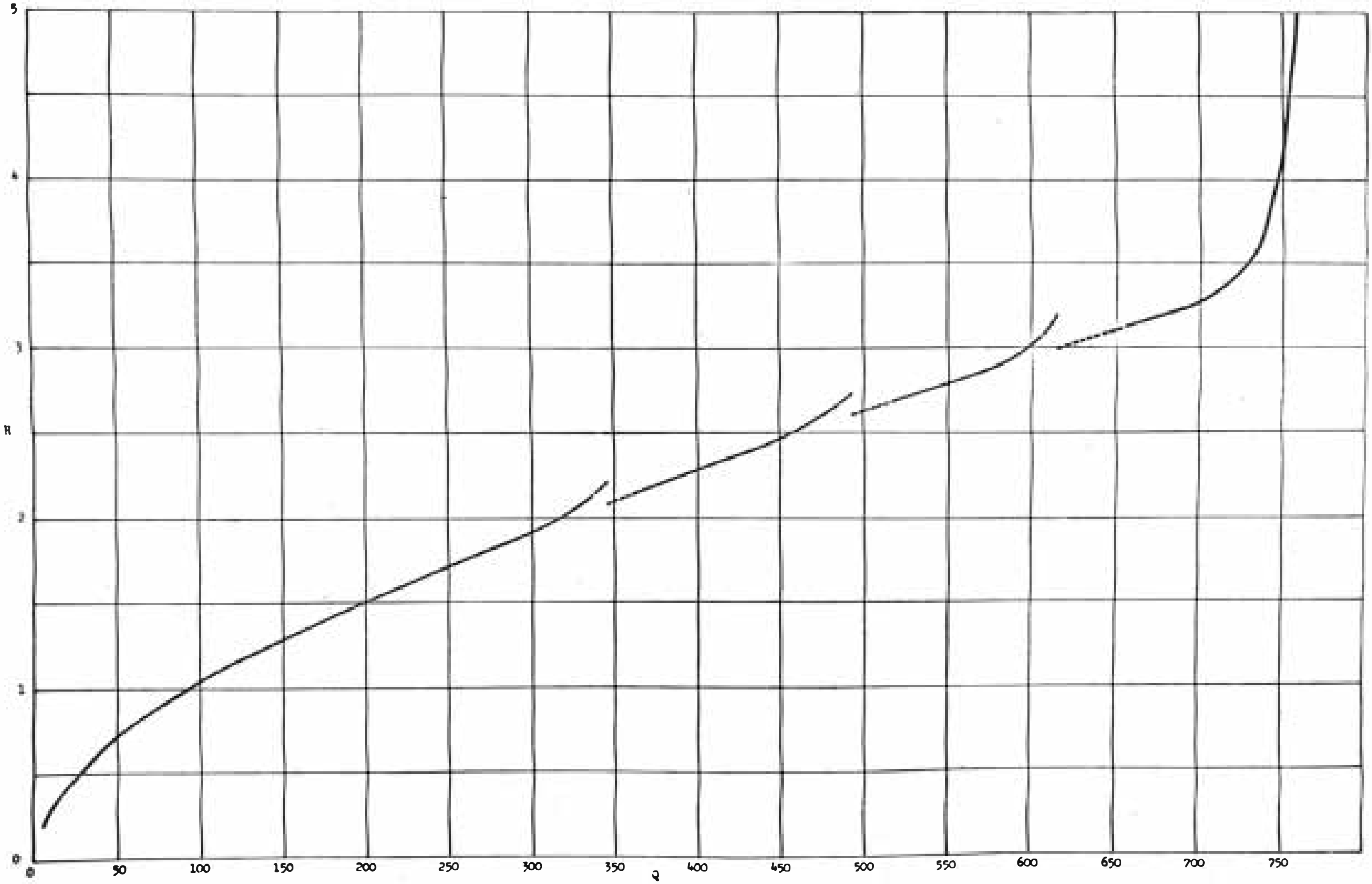


Figure XXIV. Head-Discharge Curve for Prototype Spillway controls governed the flow of the spillway. With this in mind one would foresee a desirable rating curve. This was not the case for this particular closed conduit spillway. Because three conduits were fed by one common drop inlet, full pipe flow and part pipe flow were occurring similtaneously in the spillway. With this type of flow, portions of the rating curve were indeterminate.

Referring to Figure XXIII, section-a is that portion of the curve with only partial pipe flow. Section-b is the portion of the curve with slug flow in the center conduit and part pipe flow in the outer conduits. In this section of the curve the first discontinuity takes place. Full pipe flow for the center conduit and partial pipe flow in the outer conduits takes place in section-c. The second discontinuity takes place in section-d. In this area slug flow occurs in the center conduit and one outer conduit, or slug flow occurs in one outer conduit and full pipe flow occurs in the center conduit, with partial pipe flow occurring in the third conduit in both cases. Section-e is the portion of the curve in which full pipe flow occurs in one outer and the center conduit, with partial pipe flow in the third conduit. The third, and relatively large discontinuity, occurs in section-f where slug flow occurs in one outer conduit and full pipe flow, or nearly full pipe flow, occurs for the other two conduits. Section-g is that portion of the curve in which there is full pipe flow for all three conduits.

The solid portions of the curve of Figure XXIII, are sections which were fairly well established. The broken lines are in the areas of discontinuity and a well-established curve is indeterminable.

Weir Coefficient

The equation of the weir curve for Figure XXIII was developed from the basic theoretical equation for weir flow

$$Q = CL_w H^{3/2} \tag{9}$$

where Q is the discharge in cubic feet per second, C is the coefficient of discharge, L_w is the length of the weir crest in feet, and H is the head on the crest in feet.

Because of the shape of the box inlet, the crest length (L_w) will change with a change in the head (H). The crest length was determined through the use of an equation which is used by the Soil Conservation Service for similar inlets

$$L_w = W + 1 + 2(0.4S)H \tag{10}$$

where W is the length of the flat portion of the weir, the plus 1 accounts for the 6-inch rounding, and S is the slope of the side walls, in this case 2.80. Figure XXV is a plot of head versus crest length in dimensionless form.

The results of the coefficient of discharge (C) are presented in graphical form by Figure XXVI, as derived from model data. The coefficient varies with the head (H) on the box inlet until the head is approximately 0.5D where the coefficient becomes constant.

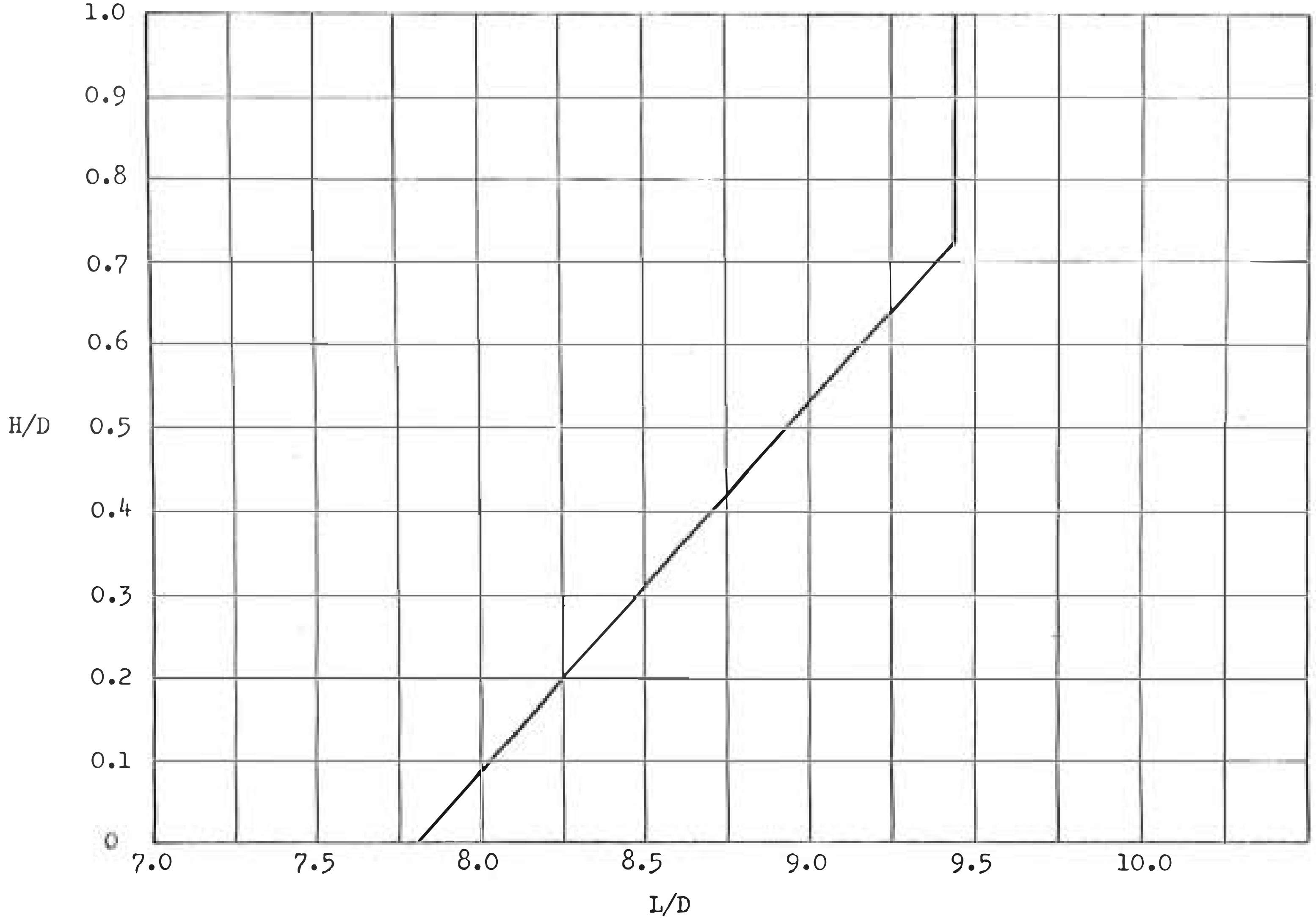


Figure XXV. Head-Crest Length Curve

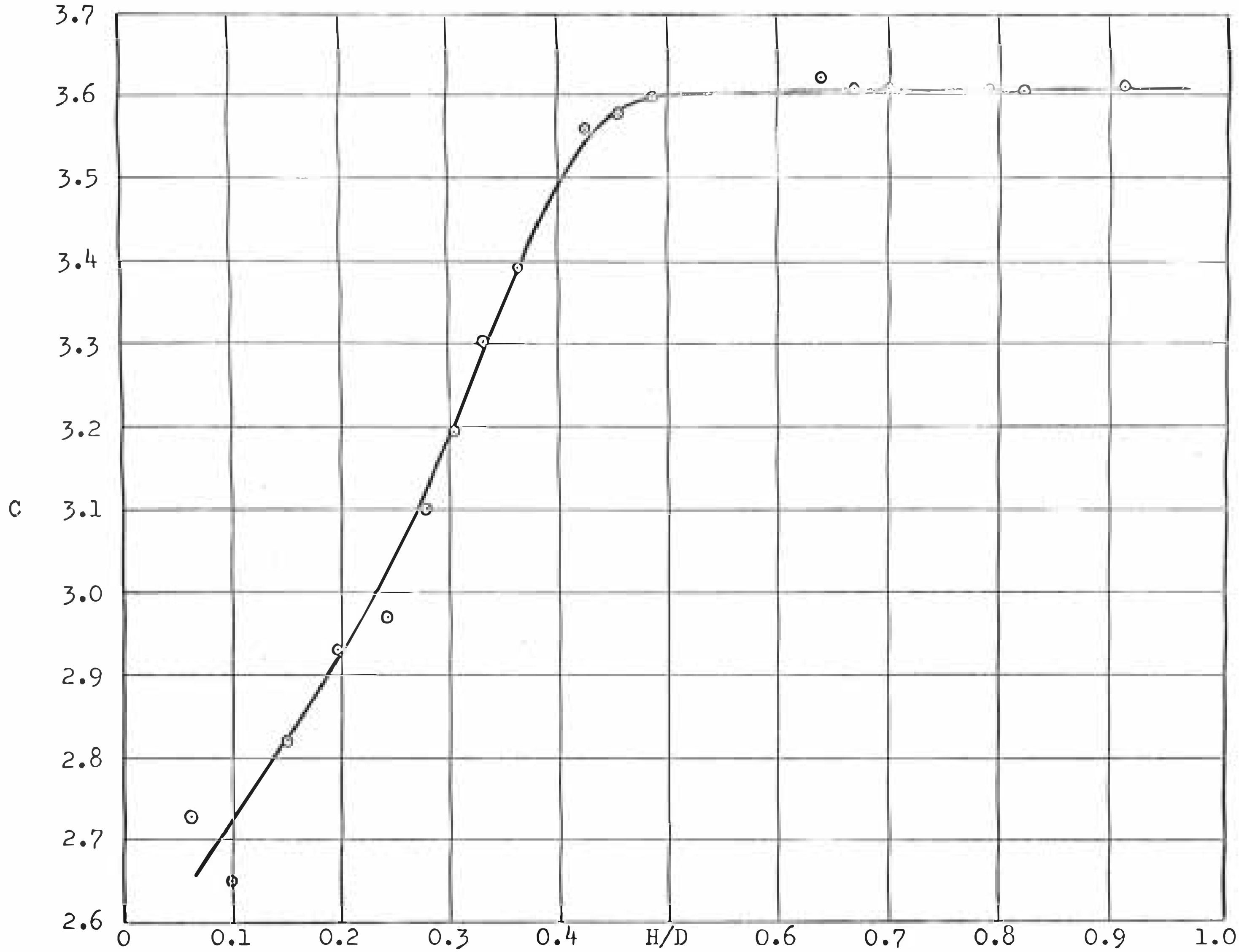


Figure XXVI. Head-Coefficient of Discharge Curve

<u>Loss Coefficients</u>

By employing the pressure data obtained from the laboratory manometers, the following average model-loss coefficients were determined

Box inlet and conduit entrance loss,	$K_e = 0.50$
Elbow bend loss,	$K_b = 0.15$
Exit loss,	$K_o = 1.00$
Conduit friction factor	$f = 0.057$

The coefficients were determined by finding the head loss (h_L) between two points with the Bernoulli equation

$$\frac{V_1^2}{2g} + \frac{P_1}{\gamma} + Z_1 = \frac{V_2^2}{2g} + \frac{P_2}{\gamma} + Z_2 + h_L \qquad (11)$$

where

V = average velocity
P = pressure
Z = elevation

and then applying the equation

$$h_L = \frac{K(V_1^2 - V_2^2)}{2g} \qquad (12)$$

A sample of the results obtained for the loss coefficients and the Reynolds number (R_n) can be found in Table 3 of Appendix B along with the method used for pressure calculations.

A combined entrance loss coefficient was determined for the box inlet and conduit entrances because the box inlet loss was negligible and difficult to determine. The calculated entrance loss coefficient was in agreement with the accepted range of coefficients for square-cornered entrances.

The calculated coefficient of 0.15 for the loss because of the bend of an elbow was in close agreement with a formula used by Donkin (11)

$$K_b = \sin^2 \theta \tag{13}$$

where θ is the angle of the bend with the horizontal.

The coefficient for exit loss as determined from the manometer data was somewhat higher than the accepted value of 1.0. Because no recovery of velocity head will occur from a pressure conduit at the exit release, it would seem logical in such an instance that it would be equal to 1.0.

The roughness of the conduits was checked with equation (11) and the Darcy-Weisbach formula

$$h_L = f \frac{L}{D} \frac{V^2}{2g} \tag{14}$$

where

h_L = head loss
f = Darcy-Weisbach coefficient
L = length of pipe
D = pipe diameter
V = velocity of fluid in pipe
g = gravitational constant

The check indicated that the conduit roughness was near the value originally determined. For all tests made on conduit roughness, the indication was that the roughness in the model was such that the Manning's "n" for the prototype ranged somewhere between 0.021 to 0.024.

Manning's "n" is related to the Darcy-Weisbach friction factor (f) by the formula

$$f = \frac{185\ n^2}{D^{1/3}} \tag{15}$$

Full Pipe Flow

The head-discharge relationship can be written for closed conduit full pipe flow. The total head (H_t) causing the flow is the head over the weir crest plus the drop through the spillway. The drop through the spillway is measured from the crest of the weir to the center of the conduit at the exit under free discharge conditions or to the tailwater elevation when the exit is submerged. The head is entirely consumed in causing water to flow through the spillway. The head consumed is given by the equation

$$H_t = \left[K_e + K_b + K_o + f\,\frac{L}{D}\right]\frac{V_p^2}{2g} \tag{16}$$

and the discharge is given by the equation

$$Q = AV_p = \frac{3\pi D^2}{4}\sqrt{\frac{2gH_t}{K_e + K_b + K_o + f\,\frac{L}{D}}} \tag{17}$$

where

D = pipe diameter
K_e = entrance loss coefficient
K_b = elbow loss coefficient
K_o = exit loss coefficient
f = Darcy-Weisbach friction coefficient
L = length of the conduit
A = area of the conduit $\frac{3\pi D^2}{4}$ since we have three conduits
g = gravitational constant
V_p = velocity in the conduit

Using equations (9) and (17) and the computed loss coefficients, the rating curve for the spillway can be drawn. The resulting discharge equation for full pipe flow of the model is found to be

$$Q = 0.95 \sqrt{H_t}$$

This equation falls within a minus three per cent of the curve predicted from the model stage-discharge data. Therefore, it seems logical to assume that the actual loss coefficients are somewhat less than measured. The correct equation which would follow the model data very closely would be

$$Q = 0.98 \sqrt{H_t}$$

which results in a full pipe flow equation for the prototype as

$$Q = 107 \sqrt{H_t}$$

Spillway Vortices

Vortices were almost entirely absent for the flow of the spillway. When the spillway attained full pipe flow, very small vortices periodically formed at the front corners of the box inlet. The condition of the vortices seemed to stay the same through the entire range of full pipe flow that was tested. At no time could the tails of the vortices be observed entering the conduits. No detrimental effect to the discharge could be detected due to the vortices.

Spillway Pressures

There are times when a knowledge of the pressures within the spillway is desirable. If a model, based on the Froude law, is run with scaled heads and discharges, the model pressures would be multiplied by the scale ratio to obtain prototype pressures. It is entirely possible for a spillway to be proportioned in such a way that pressures close to the vapor pressure of water will be obtained. Any scaled-up model pressures that are subatmospheric to the extent of about 30 feet to 33 feet of water, indicate that cavitation will occur in the prototype (27). The minimum value of pressure will ordinarily occur near the conduit entrance or at some other disturbance that lowers the pressure.

When flow velocities and pressure fluctuations are moderate or small, indicated pressures 15 feet below atmospheric are not considered objectionable. It is usually dangerous to allow pressures lower than this because unexpected surface roughnesses, vorticity and flow turbulences may momentarily lower local pressures to the cavitation range (27).

The minimum pressures in the spillway were observed in the area of the upper elbows. The minimum value obtained for the model was -1.91 feet of water. This would correspond to a negative pressure of about 20 feet for the prototype. These pressures could be considered in the dangerous range. Pressures comparable to a minus 19 feet of water for the prototype were measured in the region of the entrances to the conduits.

Future Investigation

It is the opinion of the author that a common box inlet for the three conduits may have been the primary reason for discontinuous sections of the spillway rating curve. An investigation could be carried out to eliminate these sections of the curve. A possible improvement would be to partition the existing box inlet in such a way that the flow carried by each conduit, at each instant of time, would be equal.

SUMMARY AND CONCLUSIONS

A closed conduit spillway, consisting of three conduits and one common box inlet, has been investigated to obtain information concerning its performance. The investigation was carried out on a geometrically similar model spillway. The prototype spillway consisted of a concrete box inlet and conduits made of corrugated metal pipe. The conduits were installed in such a manner as to contain three different sloping sections.

The primary objective was to obtain a head-discharge rating curve, for the spillway, for use in watershed runoff studies. Several secondary objectives were incorporated into the study. Study of the steep sloping central portion of the conduits, a common box inlet, spillway pressures, and corrugated metal pipe simulation covered the secondary objectives.

The following conclusions may be formulated from the investigation:

1. The desirable weir and pipe controls governed the head-discharge relationship for the spillway. The weir portion of the rating curve consists of a large section in which slug flow occurs in the spillway. In general the weir flow portion follows the form of the generally accepted weir flow equation, but contains three small sections where the curve is discontinuous and indeterminate. The discontinuous portions are primarily caused by the priming actions of the conduits which do not occur simultaneously. The weir flow portion of the curve follows the generally accepted equation

$$Q = CL_w H^{3/2}$$

with a variable coefficient, C, and a variable crest length, L_w. The full pipe flow portion of the rating curve is in the generally accepted form and the resulting prototype equation is:

$$Q = 107 \sqrt{H_t}$$

2. The steep sloping central portion of the conduits did not prevent the spillway from attaining full pipe flow throughout the entire length of the conduits. The large portion of slug flow of the rating curve can be partly attributed to the steep sloping section of the conduits, which causes an abrupt change in the total head, H_t, when a conduit reaches the full pipe flow region. The weir control governs the flow rate and at the same time a conduit is attempting full pipe flow.

3. The common box inlet for the three conduits may have been the primary cause for a discontinuous rating curve. Separate inlets for each conduit may have eliminated the indeterminate sections of the curve. This would be an area for future investigation.

4. The minimum pressures obtained in the spillway were approximately 20 feet of water below atmospheric pressure for the prototype spillway. This is in a range, considered by some, where possible cavitation may take place because some unforeseen roughnesses or disturbances in the conduits may lower this pressure, in instants. If slug flow or full pipe flow prevails in the spillway for any great length of time, areas within the conduits should be checked for cavitation.

5. The method of lining Lucite pipe with a small mesh hardware cloth gave a satisfactory simulation of corrugated metal pipe. Being able to line the entire conduits gave a uniform roughness throughout the conduit.

6. The over-all performance of the spillway was not satisfactory from a hydraulic point of view. All things considered, the proportions of the closed conduit spillway described in this presentation are not recommended for use under future field conditions. It is recommended that some means be employed to obtain a continuous head-discharge curve if future use of the design is considered.

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APPENDICES

APPENDIX A. DEFINITION OF SYMBOLS

Definition of Symbols

A - area of conduit

C - weir coefficient of discharge

d - average flow depth

D - diameter of pipe

e - roughness of pipe wall

f - Darcy-Weisbach friction factor

g - gravitational constant

h_L - head loss

H - head on crest of box inlet in feet

H_t - total head in feet

H_v - head above V-notch weir in feet

K_b - elbow loss coefficient

K_e - entrance loss coefficient

K_o - exit loss coefficient

L - length

L_r - length ratio (L_m/L_p)

L_w - length of box inlet weir crest

n - Manning's coefficient

P - pressure

Q - spillway discharge

R - hydraulic radius

R_n - Reynolds number

S - slope

S_f - specific gravity of manometer fluid

S_w - specific gravity of water

V - average flow velocity

V_r - velocity ratio (V_m/V_p)

W - length of flat portion of box inlet

Z - elevation

ρ - fluid density

μ - dynamic viscosity

ν - kinematic viscosity

γ - specific weight

θ - angle of elbow

subscript m - model

subscript p - prototype

subscript r - ratio

APPENDIX B. MANOMETER DATA ANALYSIS

Table 3. Sample of Loss Coefficient Results

Run	H	Q(56°F)	V	K_e	K_b	K_o	f	R_n
51	0.203	0.965	3.79	0.45 0.41			0.066 0.066	9.63×10^4
52	0.220	1.030	4.03	0.48 0.64			0.067 0.058	1.03×10^5
76	0.189	0.890	3.50	0.58 0.63			0.073	8.88×10^4
77	0.200	0.945	3.71	0.62 0.48			0.066	9.41×10^4
162	0.351	2.070	8.12		0.14 0.16	1.08 1.09	0.077	2.07×10^5
166	0.364	2.095	8.22		0.15 0.16	1.05 1.05	0.077	2.09×10^5

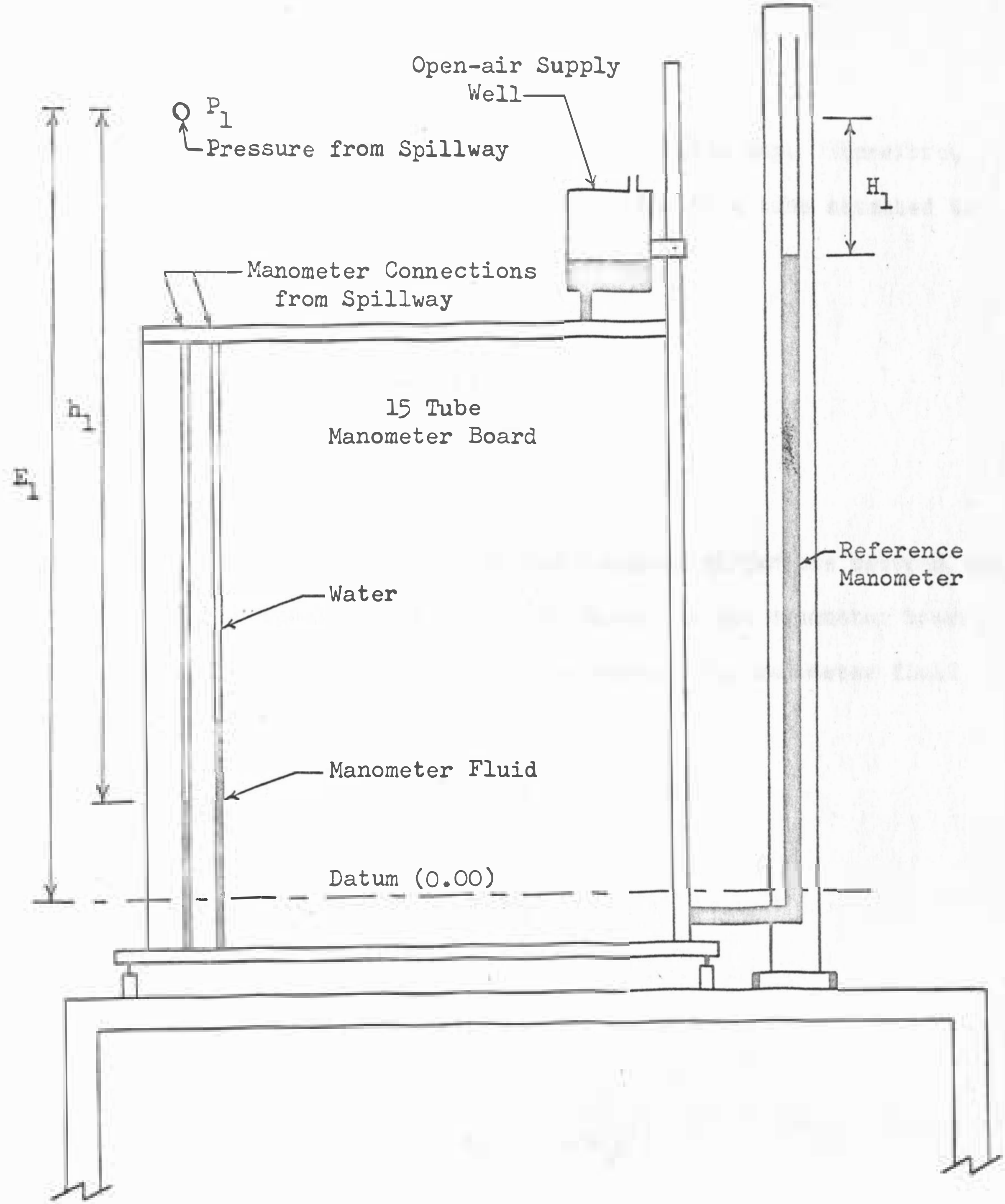

Figure XXVII. Schematic of Manometer Apparatus

From Figure XXVII it can be seen that:

$$P_1 + (S_w)(h_1) = (h_1 - H_1)(S_f)$$

$$P_1 = (h_1)(S_f - S_w) - (S_f)(H_1)$$

Where P_1 is the pressure at the specific piezometer tap. Therefore, the height to which a column of water will rise in a tube attached to the specific tap is:

$$Y_1 = E_1 + P_1$$

The total head lost to the specific tap is:

$$h_L = E_{ws} - E_1$$

All symbols are defined in Appendix A.

It can be further shown that the pressure difference between two specific taps is equal to the elevation change on the manometer board times the difference in specific gravity between the manometer fluid and water as follows:

$$P_1 = (h_1)(S_f - S_w) - (S_f)(H_1)$$

And:

$$P_2 = (h_2)(S_f - S_w) - (S_f)(H_2)$$

But:

$$H_1 = H_2 \text{ (For the same run)}$$

Therefore:

$$P_1 - P_2 = h_1 (S_f - S_w) - h_2 (S_f - S_w)$$

$$\Delta P = (h_1 - h_2)(S_f - S_w)$$

APPENDIX C. SPILLWAY DISCHARGE RESULTS

Sample Run

Run #24

Approach tank stage = 0.682 feet

4-inch orifice Δp = 1.70 inches of fluid

V-notch Weir stage = 1.363 feet

Reference points

Approach tank, lip of spillway = 0.615 feet

V-notch Weir zero = 1.050 feet

The head (H) above the box inlet lip was obtained by subtracting the reference point from the tank stage

$$H = 0.684' - 0.615' = 0.069'$$

The head above the V-notch Weir was obtained by subtracting the reference point from the Weir stage

$$H_v = 1.363' - 1.050' = 0.313'$$

This was substituted into the equation given on page 49 to obtain the discharge of the spillway

$$Q = 2.52\ H_v^{2.47} = 2.52\ (0.313)^{2.47} = 0.143 \text{ cfs}$$

This discharge was checked with the 4-inch orifice by use of the calibration curve of Figure XXVIII

Δp = 1.70 inches

Q = 0.140 cfs

The final results were then put into semi-dimensionless form to obtain

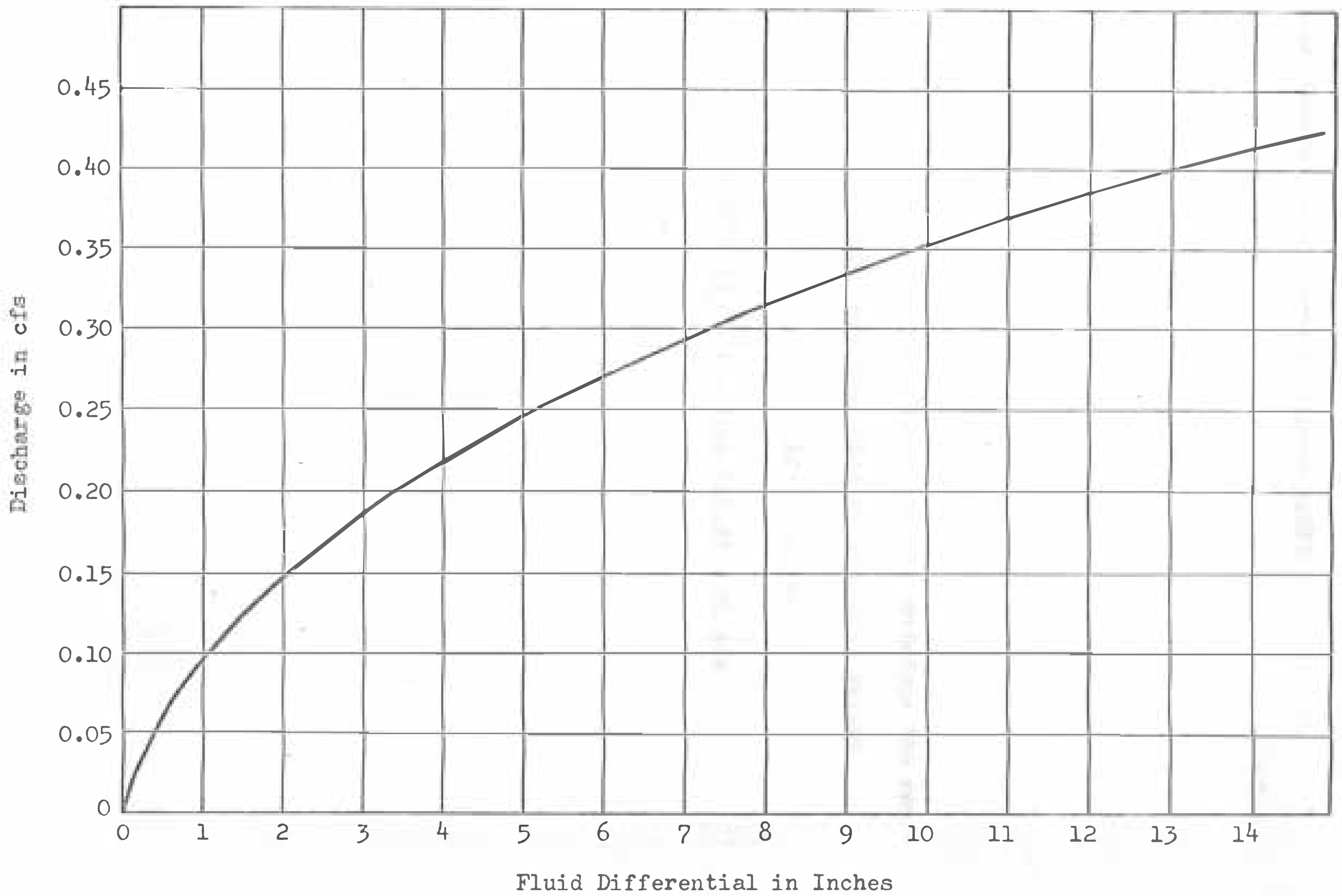

Figure XXVIII. Portion of 4-inch Orifice Calibration Curve the head-discharge curve of Figure XXIII

$H = 0.069'$

$Q = 0.143$ cfs

$D = 3.95'' = 0.329'$

$D^{5/2} = 0.062$

$H/D = 0.210$

$Q/D^{5/2} = 2.31$

To obtain the head-discharge curve for the prototype the semi-dimensionless values were converted to prototype values

$H_p = H/D\ (D_p) = 0.210\ (3.46') = 0.73'$

$Q_p = Q/D^{5/2}\ (D_p^{5/2}) = 2.31\ (22.2) = 51$ cfs

Table 4. Spillway Discharge Results

Run	H/D	$Q/D^{5/2}$	H_p	Q_p
1	0.0578	0.274	0.200	6.1
2	0.0578	0.284	0.200	6.3
3	0.0608	0.300	0.210	6.7
4	------	-----	-----	---
5	0.0669	0.376	0.231	8.4
6	0.0790	0.453	0.273	10.1
7	0.0851	0.489	0.294	10.8
8	0.0882	0.557	0.305	12.4
9	0.0942	0.605	0.326	13.4
10	0.100	0.67	0.35	15
11	0.106	0.74	0.37	16
12	0.113	0.81	0.39	18
13	0.119	0.87	0.41	19
14	0.122	0.90	0.42	20
15	0.128	1.01	0.44	22
16	0.137	1.10	0.47	24
17	0.143	1.21	0.49	27
18	0.149	1.31	0.52	29
19	0.158	1.46	0.55	32
20	0.167	1.62	0.58	36
21	0.179	1.75	0.62	39
22	0.185	1.91	0.64	42
23	0.198	2.11	0.68	47
24	0.210	2.31	0.73	51
25	0.231	2.68	0.80	60
26	0.234	2.76	0.81	61
27	0.240	2.89	0.83	64
28	0.246	3.11	0.85	69
29	0.252	3.26	0.87	72
30	0.262	3.55	0.90	79
31	0.265	3.58	0.92	80
32	0.274	3.79	0.95	84
33	0.277	4.03	0.96	90
34	0.289	4.28	1.00	95
35	0.301	4.76	1.04	106

Table 4 (continued)

Run	H/D	$Q/D^{5/2}$	H_p	Q_p
36	0.319	5.24	1.10	116
37	0.338	5.72	1.17	127
38	0.350	5.97	1.21	132
39	0.362	6.38	1.25	142
40	0.413	8.47	1.43	188
41	0.428	8.87	1.48	197
42	0.434	9.03	1.50	200
43	0.453	9.77	1.57	217
44	0.468	10.32	1.62	229
45	0.489	11.05	1.69	245
46	0.508	11.53	1.76	256
47	0.526	12.25	1.82	272
48	0.538	12.82	1.86	284
49	0.556	13.46	1.92	298
50	0.587	14.44	2.03	320
51	0.617	15.56	2.14	345
52	0.668	16.63	2.31	369
Runs 53-62 were discarded				
63	0.553	13.63	1.91	302
64	0.572	14.03	1.98	312
65	0.590	14.68	2.04	326
66	0.605	15.15	2.09	336
67	0.626	15.72	2.16	349
68	0.651	16.22	2.25	360
69	0.678	16.70	2.34	371
70	0.675	17.10	2.33	380
71	0.800	24.10	2.77	535
72	0.794	24.20	2.75	537
73	0.529	12.91	1.83	287
74	0.538	13.06	1.86	290
75	0.553	13.63	1.91	302
76	0.574	14.35	1.99	318
77	0.608	15.24	2.10	338
78	0.623	16.04	2.15	356
79	0.629	17.19	2.18	382
80	0.940	24.20	3.25	537

Table 4 (continued)

Run	H/D	$Q/D^{5/2}$	H_p	Q_p
81	0.535	12.74	1.85	283
82	0.528	12.57	1.83	279
83	0.527	11.77	1.82	261
84	0.496	11.21	1.71	249
85	0.474	10.48	1.64	232
86	0.438	9.03	1.52	200
87	0.432	----	1.49	---
88	0.428	----	1.48	---
89	0.413	7.98	1.43	177
90	0.392	7.26	1.36	161
91	0.371	6.69	1.28	148
92	0.310	4.69	1.07	104
93	0.301	4.43	1.04	98
94	0.246	3.08	0.85	68
95	0.237	2.87	0.82	64
96	0.222	2.57	0.77	57
97	0.201	2.12	0.70	47
98	0.179	1.75	0.62	39
99	0.161	1.42	0.56	32
100	0.149	1.22	0.52	27
101	0.612	15.00	2.12	333
102	0.644	15.63	2.23	347
103	0.624	16.54	2.16	367
104	0.654	17.75	2.26	394
105	0.660	18.07	2.28	401
106	0.672	18.65	2.32	414
107	0.699	19.84	2.42	441
108	0.739	21.15	2.56	469
109	0.766	21.75	2.65	483
110	0.808	22.45	2.80	498
111	0.778-0.885	22.90	2.69-3.06	508
112	0.796	23.90	2.76	531
113	0.818	24.90	2.83	553
114	0.852	26.70	2.95	593
115	0.900	27.45	3.11	610

Table 4 (continued)

Run	H/D	$Q/D^{5/2}$	H_p	Q_p
116	0.918	27.75	3.18	616
117	0.960	27.90	3.32	619
118	0.992	28.10	3.43	624
119	-----	29.15	----	649
120	0.912	29.85	3.16	663
121	0.891-0.973	28.65	3.08-3.36	636
122	-----	27.60	----	612
123	0.906	27.55	3.14	612
124	0.839	26.30	2.90	584
125	0.812	25.10	2.81	557
126	0.793	24.05	2.74	534
127	0.760	22.60	2.63	502
128	0.727	20.90	2.52	464
129	0.682	18.95	2.36	421
130	0.629	16.78	2.18	372
131	0.584	14.27	2.02	317
132	0.598	15.40	2.07	342
133	0.629	15.47	2.18	344
134	0.632-0.682	16.29	2.18-2.36	362
135	0.635	17.03	2.20	378
136	0.653	17.83	2.26	396
137	0.672	18.48	2.32	410
138	0.687	19.19	2.38	426
139	0.699	19.85	2.42	441
140	0.714	20.65	2.47	458
141	0.739	21.15	2.56	469
142	0.745	21.50	2.58	477
143	0.775	22.20	2.68	493
144	0.772-0.790	22.60	2.67-2.73	502
145	0.775	23.30	2.68	517
146	0.800	24.35	2.77	541
147	0.818	25.30	2.83	562
148	0.840	26.25	2.91	583
149	0.872	27.00	3.02	599
150	0.909	27.50	3.14	611

Table 4 (continued)

Run	H/D	$Q/D^{5/2}$	H_p	Q_p
151	0.869-0.903	27.50	3.01-3.12	611
152	0.882-0.949	28.10	3.05-3.28	624
153	0.885-0.994	28.30	3.06-3.44	628
154	0.885-0.997	28.90	3.06-3.45	642
155	-----	29.20	----	648
156	0.908	29.70	3.14	659
157	0.921	30.55	3.19	678
158	0.933	31.30	3.23	695
159	0.949	32.05	3.28	712
160	0.961	32.50	3.32	721
161	0.997	32.85	3.45	729
162	1.065	33.40	3.68	742
163	1.230	33.95	4.26	753
164	1.345	34.15	4.65	760
165	1.465	34.30	5.07	761
166	1.105	33.85	3.82	752
167	1.035	32.85	3.58	729
168	0.964	32.55	3.33	722
169	0.924	30.30	3.20	673
170	0.815	24.95	2.82	553
171	0.678	18.87	2.34	418
172	0.535	12.66	1.85	281
173	0.568	13.79	1.97	306
174	0.587	14.36	2.03	318
175	0.605	14.84	2.09	330
176	0.617	15.24	2.14	338
177	0.602-0.638	15.65	2.08-2.21	347
178	0.621	16.29	2.15	361
179	0.629	16.77	2.18	372
180	0.638	17.02	2.21	378
181	0.657	17.75	2.28	394
182	0.672	18.39	2.32	408
183	0.687	19.20	2.38	426
184	0.705	19.84	2.44	440
185	0.712	20.25	2.46	449

Table 4 (continued)

Run	H/D	$Q/D^{5/2}$	H_p	Q_p
186	0.723	20.65	2.50	458
187	0.736	20.95	2.55	465
188	0.754	21.35	2.61	472
189	0.738	21.10	2.55	468
190	0.757	21.50	2.62	477
191	0.744-0.768	21.85	2.57-2.66	485
192	0.751	22.20	2.60	493
193	0.760	22.50	2.63	499
194	0.778	23.15	2.69	514
195	0.784	23.65	2.71	525
196	0.798	24.20	2.76	537
197	0.805	24.70	2.79	548
198	0.808	24.75	2.80	549
199	0.814	25.10	2.82	557
200	0.827	25.55	2.86	567
201	0.842	26.20	2.91	582
202	0.848	26.65	2.94	592
203	0.853	26.65	2.95	592
204	0.860	26.85	2.98	596
205	0.881	27.15	3.05	603
206	0.875	27.10	3.03	602
207	0.890	27.30	3.08	606
208	0.903	27.45	3.12	609
209	0.881-0.917	27.85	3.05-3.17	618
210	0.884-0.963	28.15	3.06-3.33	625
211	-----	-----	----	---
212	0.906	29.80	3.14	661
213	0.912	29.90	3.16	663
214	0.915	30.15	3.17	669
215	0.921	30.30	3.19	673
216	0.929	30.85	3.21	685
217	0.936	31.30	3.24	695
218	0.942	31.45	3.26	698
219	0.952	31.90	3.29	708
220	0.963	32.45	3.33	720

Table 4 (continued)

Run	H/D	$Q/D^{5/2}$	H_p	Q_p
221	0.975	32.65	3.37	724
222	0.997	32.75	3.45	727
223	1.034	32.95	3.58	731
224	1.076	33.40	3.72	742
225	1.250	33.90	4.32	752
226	1.380	34.10	4.77	757
227	1.105	33.40	3.82	742